FINAL DRAFT

Peer Review Report

London Avenue Canal I-wall Load Test

Prepared for

Hurricane Protection Office

New Orleans District

U.S. Army Corps of Engineers

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EXECUTIVE SUMMARY

The Interagency Performance Evaluation Taskforce (IPET) study that was initiated after Hurricane Katrina concluded that for some of the flood control levees in New Orleans a gap could develop between the I-wall and the levee soils on the canal side due to excessive hydrostatic pressure on the I-walls. Once formed, the gap could allow a preferential seepage path along the I-wall/levee interface, resulting in high water pressures on the landside of the levee. The U.S. Army Corps of Engineers (USACE) proposed to perform a load test of a portion of the London Avenue Canal to better understand the mechanism regarding development and propagation of a gap along the I-wall/levee interface due to hydrostatic loading. The USACE proposes to use information from this load test to evaluate whether the Safe Water Elevation (SWE) in the canal can be raised from the present post-Katrina approved level of El +4 North American Vertical Datum (NAVD) to some higher level. The Southeast Louisiana Flood Protection Authority (SLFPA) requested an independent geotechnical evaluation of the load test program proposed by USACE to confirm (i) that the test can be performed without causing functional failure of the wall; and (ii) that data from the test will be a reliable basis for establishing a SWE. This report presents the results of the requested independent geotechnical evaluation.

The independent review by the authors concluded that the USACE-proposed load test program provided several redundant safeguards and that the proposed load test program could be safely conducted without jeopardizing the integrity of the existing levee. Furthermore, the authors concluded that information from the load test program could be used to evaluate the SWE in the London Avenue Canal. The authors noted that studies performed by USACE and others after publication of the IPET Report concluded that seepage beneath the levee, in the absence of a gap, could also result in the failure of the levees along the London Avenue Canal. Technical review results presented in this report acknowledge and concur with the two identified potential failure modes. The authors offer recommendations for performing the hydrostatic load test on the canal to obtain information during the load test regarding the development and progression of the gap, as well as under-seepage beneath the levee.

The authors concluded that performing the load test in two stages could enhance the proposed load test. As proposed, the first stage would evaluate the development and propagation of the gap, while the second stage will focus on under-seepage beneath the levee. Load test configuration modifications proposed by USACE facilitate the evaluation of the second stage of testing by extending the width of the cofferdam used for the load test program by only 10 ft into the canal and installing infiltration pipes into the poorly graded sand layer underlying the bottom of the canal. The authors concur with this USACE-proposed modification and further recommend that the depth of the cofferdam sheetpiles be extended into the clays underlying the sand to facilitate data evaluation during the load test. The authors recommend that several additions/modifications be made to the original program prior to commencement of the hydrostatic load test program. These changes include: (i) obtaining additional strength and stiffness characterization data in the subsurface soils on the landside of the levee; and (ii) focusing data collection activities during the load test on pore pressures in the underlying sands in lieu of monitoring physical deformations and earth pressures related to the I-wall gap. During load testing, the authors recommend that the originally proposed load testing protocols be adjusted to allow stabilization of the pore water pressure readings within pre-determined limits prior to incrementally increasing the hydrostatic loads. Finally, the authors recognize a significant benefit in having the field-collected monitoring data collected and analyzed in real time during the load test as originally proposed by USACE. In addition, the authors believe that it would be beneficial to have individuals from USACE and Virginia Tech who were responsible for the post-Katrina London Avenue Canal numerical analyses maintain an on-site presence during the performance of the load test so that their analysis models can be updated in real-time using the actual performance monitoring test results provided by the instrumentation network.

The SLFPA requested that the technical review also include comments regarding the potential for extending the load test results to allow an evaluation of the SWE in the entire London Avenue Canal. The authors conclude that results of the two-stage hydrostatic load test, coupled with the recommended additional subsurface characterization tests performed along the London Avenue Canal and pore water pressure monitoring results along other reaches of the canal during service operations, can be used to assess the reasonableness of any proposed increase in the SWE in the London Avenue Canal. Recommendations regarding the extension of the load test results to the determination of a SWE are provided.

Finally, the authors have provided the summary and recommendations presented in this report to the SLFPA and USACE for review and comment. In many cases, some of the authors' comments and recommendations have been verbally acknowledged and reportedly included in revisions to the previously provided documentation. As appropriate, in this final report, the authors will acknowledge the reported revisions that will be incorporated into the load test of the London Avenue Canal.

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1. INTRODUCTION

The U.S. Army Corps of Engineers (USACE) proposed to perform a load test of a portion of the London Avenue Canal (canal) to evaluate a specific mechanism regarding the development and propagation of a gap along the I-wall/levee interface due to hydrostatic loading. The purpose of the London Avenue Canal I-wall/levee load test is to better understand and/or confirm the mechanism regarding gap development/propagation and to evaluate whether the Safe Water Elevation (SWE) in the canal can be raised from the present post-Katrina approved level of El +4 North American Vertical Datum (NAVD) to some higher level. The USACE's Engineering Research and Development Center (ERDC) set the revised SWE, following issuance of Volume V of the Final Draft Report [(1)] of the Interagency Performance Evaluation Task Force (IPET) dated June 1, 2006. The IPET Report was prepared based on an assessment of the I-wall and levee performance during Hurricane Katrina. The proposed I-wall/levee load test is planned to be centered on about General Design Memorandum [(2)] (GDM) Station 108+75 of the east levee, where stationing is defined along the center line of the east levee. The load test will incorporate five monoliths of the wall, each approximately 30 feet long, for a total test section length of approximately 150 feet. A rectangular cofferdam will be constructed on the canal side of the I-wall/levee test section and water will be pumped into the enclosure to raise the water level against the wall in 6-inch vertical increments to a maximum El +7.5 NAVD. Instrumentation will be provided to monitor pore pressures beneath the levee on the protected side (i.e., landside) of the levee and to evaluate wall and ground deflections on both sides of the I-wall on a real time basis.

The Southeast Louisiana Flood Protection Authority (SLFPA) requested that an independent technical evaluation be performed of the proposed load test program and that results of the review be provided to the SLFPA before the load test commences. USACE retained Dr. Robert Bachus, P.E., of Geosyntec Consultants and Dr. Ray Martin, P.E., of Ray E. Martin, LLC to provide the technical review on behalf of SLFPA. The technical review is intended to focus on whether the proposed load test program: (i) will provide the requisite information that will allow a decision to be made regarding the SWE in the canal; and (ii) can be safely performed without jeopardizing the integrity of the existing I-wall/levee system.

This report presents the collaborative opinions of Drs. Bachus and Martin (the authors) as to the technical feasibility of the load test and provides recommendations regarding suggested modifications to the original load test program to enhance the information recovered during the load test. The reader is referred to Volume V of the IPET Draft Final Report referenced previously for a general background on the geology, characterization of the soils in the New Orleans area, and for a detailed assessment of the conditions in the vicinity of the north breach failure of the London Avenue Canal I-wall/levee. References reviewed by the authors in preparing this report are included in Appendix A. Following this introduction, the remainder of the report is organized to present: (i) a brief summary of IPET analyses and conclusions; (ii) a description of the original load test program; (iii) a review of available subsurface data; (iv) a review of post-IPET load test studies; (v) a discussion of an alternative failure mode; (vi) suggested modifications to the load test program; and (vii) a summary of recommendations proposed by the authors.

2. SUMMARY OF IPET DATA

The beginning of this section presents a summary of the local and regional geology and a discussion of the design of the I-walls. Site-specific reference is made to the “north breach” of the London Canal, as this area was selected for the load test. Using information presented in the IPET Report [1], a summary is presented herein of the causes of: (i) the north breach failure located on the west side of the canal; and (ii) the I-wall excessive deflections located on the east side of the canal in the vicinity of the north breach. The subsurface conditions at the locations where the I-wall/levee was breached/deflected are similar to the stratigraphy at the load test section, as the proposed test section is located immediately to the south of the north breach area.

2.1 LOCAL AND REGIONAL GEOLOGY

The following summary of the geologic conditions in the New Orleans area was developed from review of the IPET Report Appendix 1 [1]. The levee for the canal is located over a sequence of recent sediments that were deposited over the past 7,000 years. This recent natural fill overlies an old Pleistocene surface that was exposed at the time the last ice age ended and the sea began to rise (i.e., approximately 15,000 to 12,000 years before the present). At that time, the sea was about 300 ft lower than its current elevation and the Gulf of Mexico shoreline was located much farther to the south than it is today. The ancestral Mississippi River valley was to the west of New Orleans in the area of Morgan City, LA. As the sea began to rise to the present level, New Orleans (and the test site area) was filled with Holocene deposits consisting of (from bottom to top) bay/sound/estuarine clays, sands (specifically the Pine Island Barrier Beach sands), lacustrine clays, and finally organic clay, peat marsh, and swamp deposits. One of five identified delta complexes in this area of southern Louisiana, the St Barnard delta complex, containing the Mississippi River and its distributary channels, were responsible for depositing sediments in this area of New Orleans.

The local sediment deposition process included the following specific stages. On top of the Pleistocene clays at depth, the underlying bay/sound/estuarine clays were deposited as the sea level began to rise rapidly and inundate the area. The Pine Island Barrier Beach formation represents the sand layer that was deposited about 4,000 to 5,000 years before present when the sea level was about 10 to 15 feet below current elevations. In some areas along the canal alignment, Holocene lacustrine clays were deposited in a fresh water environment on the south side of the barrier beach that formed a shoreline before the Mississippi River delta advanced into the Gulf of Mexico. The recent surficial swamp/marsh/organic clays (called marsh clays in this report) that overlie the sand were deposited as recently as 500 years ago on the ground surface formed by previous flooding of the Mississippi River and the Bayou Sauvage distributary channel, referenced locally as Bayou Metairie. This channel was located along the south edge of the older Pine Island Barrier Beach just to the south of the test section as shown on Figure B-1 (3) of Appendix B.

2.2 <u>I-WALL DESIGN</u>

The I-wall was constructed in 1996 and replaced an existing sheetpile wall. The I-walls were intended to provide improved protection of property immediately adjacent to the I-wall levee during periods of high canal water level. Portions of the I-wall profile and section taken from Construction Drawings provided to the reviewers and presented herein as Figures B-2 and B-3 (3) in Appendix B. Prior to construction of the I-wall, the top elevation of the existing sheetpile wall was approximately El +11.0 NAVD. The datum on the referenced drawings was National Geodetic Vertical Datum (NGVD) and the conversion factor to NAVD is -1.5 feet (i.e., NAVD = NGVD-1.5 feet) according to a personal communication from USACE and Dr. Tom Brandon of Virginia Tech (4), a technical consultant to USACE on this project. The existing sheetpile wall was cut off at about El +2 NAVD during construction of the new concrete I-wall. A third sheetpile wall had been located between the old sheetpile wall and the new I-wall. These sheetpiles were reportedly cut off and pulled during I-wall construction. The top of the I-wall is El +12.9 NAVD and the base is El +0.5 NAVD, resulting in a concrete I-wall height of approximately 12.4 ft. Finished grade at the top of the levee is shown as El +2.5 NAVD, resulting in an exposed I-wall height of approximately 10.4 ft. The wall is supported on a PZ-22 sheetpiling driven to a top elevation of El +3.25 NAVD. Thus the embedment of the sheetpiling in the base of the I-wall is approximately 2.75 ft. The sheetpiling extended to El – 21.5 NAVD into the underlying sand.

2.3 FAILURE AND DEFLECTION OF NORTH I-WALL SECTIONS

The I-wall failed at two locations within the London Avenue Canal during Hurricane Katrina. These failures have been designated the south breach and north breach in the referenced IPET Report (1). The north breach was located just south of the Robert E. Lee Bridge on the west levee and was centered at about GDM Station 114+75. At this approximate station location (i.e., centered at GDM Station 118+00), a portion of the I-wall on the eastern side of the levee tilted outward and was severely permanently deflected by the high water in the canal, but did not result in a breach. The maximum water level in the canal during Hurricane Katrina at the north breach was estimated to be between El +8.2 and +9.5 NAVD (1). It is noted that the proposed location of the load test is immediately south of where this I-wall deflection occurred on the east levee.

Regarding the cause of failure, IPET (1) made the following statement on page V-52 of the IPET Report.

> *"Field evidence, analyses, and physical model test show that the breaches were due to the effects of high water pressures within the sand layer beneath the levee and I-wall, and high water loads on the walls. The---formation of a gap between wall and the levee fill on the canal side of the wall--- allowed high water pressures to act on the wall below the surface of the levee, severely loading the wall. --- an additional effect of the gap was that was that water flowed down through the gap into the underlying sand. High water pressures in the sand*

uplifted the marsh layer on the landside of the levee, resulting in concentrated flow and erosion, removing material and reducing support for the floodwall."

IPET further discussed the reduced support for the floodwall more specifically on page V-42 of the IPET Report.

> *"It seems likely that the failure and breach were the result of insufficient passive resistance---"* and that *"---the passive resistance was likely reduced by the effects of water seeping through the foundation soils beneath the levee and the marsh layer inland, inducing uplift pressures and reduced shear strengths."*

IPET also concluded on page V-43 of the IPET Report that steady state seepage through the underlying sand was established quickly as the canal water level rose. IPET also noted that the pore pressures and uplift pressures at the base of the marsh clays was essentially not affected by the presence of the clays as long as the marsh clays had a permeability of at least two orders of magnitude less than the sand.

With respect to the deflection of the I-walls across from the north breach, IPET concluded the following on page V-52 of the IPET Report.

> *" ---it seems reasonable to assume that the wall---must have been close to failure, but this location has not been analyzed in detail."*

The IPET Report also noted that sinkholes formed on the canal side of the I-wall and sand boils formed on the protected side of the I-wall/levee. The report titled *London Avenue Canal Load Test – Small Load Test Site Selection*, by Vroman, Brandon and Schwanz [(4)] also noted that heaving was reported of the ground on the protected side of the levee.

2.4 IPET ANALYSIS OF I-WALL DEFLECTION

IPET [(1)] concluded that a gap formed between the I-wall sheetpiling and the embankment during the failure of the west I-wall at the north breach and that a similar gap may have developed at the deflected I-wall on the east side levee across from the breach. This gap is believed to have formed and increased in width and depth as the I-wall rotated outward under the pressure imposed by the water level in the canal. In the case of the north breach, a "half cracked" failure model (half-cracked model) was assumed. Figure B-4 [(1)] of Appendix B illustrates the model. The model was verified by a soil-structure interaction (SSI) analysis using the computer program PLAXIS and the results are presented and described in Appendix 9 of the IPET Report Volume V [(1)]. According to the assumptions of the half-cracked model, the gap extended from the ground surface downward to the interface of the marsh clays and the underlying sands. A 1-foot thick marsh layer was assumed to exist in the bottom of the canal in the IPET analyses. The IPET analysis was performed using two different stiffness values (i.e., average stiffness and increased stiffness) for the levee embankment and marsh clays to assess the sensitivity of the gap development and migration to soil stiffness.

The results of the IPET analysis indicated that a crack was initiated when the water pressure on the canal side exceeded the horizontal effective stress for both soils of average stiffness and increased stiffness. This condition occurred when the water in the canal reached the crest of the levee at El +4.4 NAVD. For average stiffness soils, the initial gap (also referenced as the "crack") extended to El -5 NAVD and for increased stiffness soils, the initial crack penetrated to El -3 NAVD. As the water level in the canal increased, the crack penetrated deeper in the levee until it reached the top of the sand (i.e., El -12.9 NAVD in this case). Calculation results indicated that the full penetration of the gap to the top of the sand occurred at a canal water level El +6 NAVD for the soils of average stiffness and at El +8 NAVD for the stiffer soils. Most significantly, the analysis model indicated that once the crack or gap opens to the top of the sands the pore pressures increase fairly rapidly in the sand because of the shortened seepage path. The increase in pore pressure resulted in a decrease in the effective vertical stress. When the calculated effective stress is reduced to zero, there is a loss of stability and a resulting increase in lateral deformation of the wall.

2.5 THE LOAD TEST CONCEPT

The reported failure mechanism (i.e., gap development and migration) led USACE to conclude that a large-scale load test could be safely performed in the London Avenue Canal to better understand the gap development mechanism and to evaluate the SWE within the canal. It is noted that following the north and south breaches, repairs were initiated by USACE using a more robust L-wall section; the remainder of the canal continues to be protected by I-wall.

3. ORIGINAL PROPOSED LOAD TEST PROGRAM

The following sections discuss the features of the proposed load test as presented to the reviewers for consideration.

3.1 COFFERDAM CONFIGURATION

The test section cofferdam for the USACE-proposed load test was planned to have a rectangular configuration 150 feet in length parallel to the I-wall and with two 25-foot long wing walls tied into the I-wall on the eastern side of the canal. A drawing illustrating this configuration in plan view is included as Figure B-5 [(3)] of Appendix B. Cross sections of the cofferdam perpendicular to the I-wall, shown in Figures B-6, B-7 and B-8 of Appendix B, illustrate the position of the cofferdam sheetpiling on the slope of the embankment. These figures indicate that the area exposed to canal water during the test would be restricted to the embankment fill slope and perhaps a portion of the cut slope of the marsh clays. Water within the cofferdam would not have direct access to the sand in the bottom of the canal.

The sheetpiling for the cofferdam is proposed to be driven to tip El -31.5 NAVD and thus will only penetrate about 60 to 65 percent of the sand layer. The bottom of these sands is estimated to be about El -44 to -45 NAVD at the test section. The cross sections of Figures B-6, B-7 and B-8 incorrectly illustrate fully penetrating sheetpiles for the originally proposed load test. Figure B-18 [(11)] of Appendix B correctly reflects the partially penetrating sheetpiles included in the original load test program.

The hydraulic, structural, and foundation support pile designs of the cofferdam have been prepared by USACE and their consultants. The authors did not perform a detailed review of these design reports, as the scope of work requested by the SLFPA related only to geotechnical issues. .

3.2 TIME RATE OF LOADING

In the USACE-proposed load test program, the I-wall will be loaded by increasing the depth of water in the cofferdam in 6-in. increments. As proposed, each load will be held for 4 hours before adding the next increment of water. Gates are included in the cofferdam to facilitate drainage if the I-wall or the levee should show any sign of excessive movement that may represent a precursor to failure. A rubber containment dam is also planned on the protected side of the test section in the event of an I-wall/levee failure as a means of providing a degree of redundant protection.

3.3 INSTRUMENTATION

Instrumentation for the project was designed by URS and is described in a May 2, 2007 report titled *Design of Automated Data Acquisition System (ADAS)* [5]. Instruments proposed by URS for the load test program include surface water level monitors within the cofferdam, protected side open standpipe piezometers penetrating into the sands of Stratum D, tilt meters on the I-wall, crack monitors across I-wall monolith joints, gap monitoring devices, and inclinometers. The instruments will be read by an ADAS with manual back up. Instrument thresholds during the load test will be established to set off alarms so that the water in the cofferdam may be drained into the adjacent canal in the event of any indication (i.e., increased deformation and/or increase in pore pressure) that could indicate the potential for failure. The instrumentation (and protection) system has several levels of redundancy. A plan and cross section of the proposed instrumentation system is included in Figures B-9 and B-10 [5] of Appendix B.

3.4 CONCLUSION AND RECOMMENDATION

The authors believe that the original load test program was designed to adequately fulfill the requirements to test the gap theory and that the proposed program will provide valuable data to help better understand this mode of failure. Many redundant features have been built into the test procedures and monitoring to assure a high degree of protection against a failure of the test section during the test. However, as discussed in later sections of this report, it is recommended that the test be expanded in scope to evaluate an alternate failure mode that has been identified in the USACE project documents..

4. AVAILABLE SUBSURFACE DATA

Before considering a possible alternative mode of failure, it is appropriate to discuss the subsurface conditions and soil properties of the test section. Subsurface data developed for design of the I-walls and post-Katrina field investigations in the area of the test section were reviewed. Laboratory and insitu test data from both the I-wall design phase and subsequent studies are summarized in the following sections.

4.1 SITE TOPOGRAPHY AND FIELD INVESTIGATION

The test section extends from about GDM Station 108+00 to 109+50 with the center line at GDM Station 108+75 as shown on the Boring Location Plan of Figure B-11 [(3)] of Appendix B. Photographs of the site are presented in Figures B-12 through B-15 of Appendix B. The canal is about 120 feet wide with the water surface at El 0 NAVD. The GDM stationing commences from the south and increases to the north. The three cross sections lines shown on Figure B-11 are located at about GDM Stations 108+50, 108+75 and109+00 based on the coordinates listed with the ground surface survey data provided [(3)]. These ground surface cross sections are plotted on the Subsurface Profiles of Figures B-6 through B-8 [(3)] of Appendix B and were labeled in the data provided for review as *Test Section, 150 Feet North of Test Section and 150 Feet South of Test Section*. Based on the ground surface survey grades for these cross sections, they appear to be located at the stations noted above: GDM Stations 108+50, 108+75 and109+00. (Note: Based on the scale of the plan view, it appears that the above referenced title should reflect cross-sections located +/- 25 ft from the centerline of the test section.)

The locations of borings used to establish the stratigraphy at the test site are shown on Figure B-11 [(3)] of Appendix B and include conventional borings and geoprobe soundings. Boring B-59 and B-91 [(2)] were drilled in 1985, while the geoprobes were drilled in May 2007. The geoprobes drilled in the canal are labeled Borings S1LCGP–1 through S1LCGP-4 and those drilled on the protected side are labeled Borings LKGSC-1 through LKGSC-3 (levee crest locations) and LKGST-1 through LKGST-3 (levee toe locations).

Only one geoprobe was drilled in the canal within the test section. Borings B-59 and B-91 were drilled at GDM Station 109+75 just to the north of the test section. Boring B-59 was drilled to a depth of 50 feet from the levee crest and penetrated the bay/sound/estuarine clays just above the Pleistocene clays and Boring B-91 was drilled at the canal centerline to a depth of 10 feet below ground surface with 10 feet of water in the canal. The geoprobes extended to depths of 27 to 40 feet. The lacustrine clays were encountered only in geoprobe Boring LKGSC-1, the barrier beach sands were encountered in all of the geoprobe borings and the bay/sound/estuarine clays were encountered only in geoprobe Boring LKGST-2. In addition to these borings, Borings, B-57 and B-58[2] were drilled south of the test section at Stations 102+95 and 104+75, respectively, on the levee and were used to interpolate the surface of the bay/sound/ clays.

The cross sections shown in Figures B-6 through B-8 [3] in Appendix B indicate the crest of the levee is at about El +2.5 NAVD. The ground surface grade at Boring B-59, as noted on the log, is +3.5 NGVD or El +2.0 NAVD. The ground surface grades at the geoprobe boring locations on the levee crest are noted on the logs as being about El +3.5 NAVD. Based on visual assessment of current site conditions, this elevation of +3.5 NAVD appears to be high and may actually reference an NGVD elevation. The canal bottom at the cross section locations is between El -10.9 and -11.4 NAVD. The protected side grades range from El -5 NAVD at the levee toe to El -7 NAVD at Warrington Drive east of the canal.

New stationing has apparently been established and the centerline of the load test corresponds to new Station 18 + 50. This stationing extends from the north and increases to the south. The new stationing was used to identify the locations of the protected side geoprobes on the logs provided for review [3]. The locations of the canal side geoprobes and Borings B-57, B-58, B-59 and B-91 were identified on the logs by the old stationing. These borings and the referenced stationing were used to develop the surface topography used to present subsurface stratigraphy (subsequently discussed) of the test site.

4.2 STRATIGRAPHY AND SOIL PROPERTIES

The stratigraphy at the test site has been divided into five strata, based on the borings discussed above. A description of the five strata is provided below. The soil properties from the boring logs are also summarized. The classifications shown in parentheses are based on laboratory test results on specimens obtained from the test section [2, 3].

- Levee fill – These soils are described typically as stiff to very stiff consistency fat clays (CH) with sand pockets, wood fragments, and roots. The levee fill extends from the ground surface at about El +2.5 to -5 NAVD for a total thickness of approximately 7.5 feet at the thickest location.
  - Moisture content values ranged from 25 percent near the top of the levee to 58 percent at the base of the levee.
  - Liquid Limit (LL) values ranged from 56 to 62 percent and the Plasticity Index (PI) values ranged from 32 to 41 percent.
  - Dry density values ranged from 76.5 to 86.6 pcf and wet density values range from 107.0 to 115.9 pcf.
  - Undrained shear strength values for four specimens were 432 and 1,728 at a sampling depth of 2 feet and were 682 and 705 psf at a sampling depth of 5 feet.
- Marsh clay – These soils are described as generally very soft to soft consistency organic clays (OH) with wood fragments and roots. In proximity of the load test area, the surface grades of this stratum vary as follows.
  - Under levee - El -4 to -5 NAVD.
  - At protected side toe of levee on ground surface - El -5 NAVD.

- In Canal in Borings B-91, S1LCGP-2 and S1LCGP-4 – El -11 NAVD.

The bottom of the layer ranges from about El -8 to -12 NAVD below the test section for a total thickness of about 4 to 5 feet under the levee, about 5 to 7 feet at the toe of the levee and 0.5 to 3 feet in portions of the canal. Typically, the marsh clays are thinner under the levee due to consolidation under the weight of the levee fill. The thin layer of marsh clay in the canal did not exist at the one location within the test section, as indicated by geoprobe Boring S1LCGP-3.

- Moisture contents.
    - 172 to 266 percent under the levee.
    - 54 to 109 percent at toe of levee and beyond (excluding desiccated surface).
    - 84 to 203 percent in the canal.

- Lacustrine clay – These soils were only encountered in geoprobe Boring LKGSC1 and consisted of soft consistency fat clays (CH). The surface grade of this layer was EL -10 NAVD and it extended for 2 ft to El -12 NAVD.
    - Moisture content - 74 percent.
    - LL and PI - 108 and 76 percent, respectively.
- Barrier beach sand – These loose to medium density poorly graded sands (SP) and silty sands (SM) underlie the marsh clay; in the area of geoprobe Boring LKGSC-1, this layer underlies the lacustrine clay. The formation extended to El -45.0, -44.5 and -40.5 NAVD in Borings B-59, LKGST-2 and B-57, respectively, and is about 34 feet thick in the test section area.
    - Standard Penetration Test (SPT) N-values from Borings B-58 and B-59.

- 4 to 7 underlying the marsh soils of Stratum B to El –22 NAVD.
- 12 to 26 in the sands to El -39 NAVD.
- 4 to 10 in the sands to the base of the stratum.

- D_{10} size ranged from 0.088 to 0.130 mm with a an average value of 0.106 mm, which reportedly could be correlated to a permeability of about $1.4x10^{-2}$ cm/sec.

- Bay/sound/estuarine clay – These clays underlie the barrier beach sands and lie on top of the Pleistocene clays. They were only encountered in Borings B-59 and LKGST-2 and consisted of soft to stiff consistency fat clays (CH). At the test section these clays are estimated to be about 10 to 20 feet thick.
  - Moisture content values ranged from 43 to 65 percent.
  - Dry and wet density values ranged from 60.5 pcf and 100.0 pcf, respectively.
  - Undrained shear strength fro a single specimen was 908 psf

The Pleistocene formation was not encountered by any of the test borings at the test site.

5. POST-IPET LOAD TEST STUDIES

After publication of the IPET Report, USACE initiated and/or commissioned several studies to evaluate the implications of the IPET findings on the operation of the flood protection canals in New Orleans, including the operation of the London Avenue Canal. Some of these analyses provided essentially an independent assessment of the IPET findings. The results of these studies were provided to the authors for review as part of the preparation of this report.

5.1 LOAD TEST SITE SELECTION PROCESS

The proposed test section was evaluated in an undated report titled *London Avenue Canal Load Test – Small Load Test Site Selection* by Vroman, Brandon, and Schwanz [(4)]. This report assessed the subsurface conditions along the entire length of the London Avenue Canal and identified three possible sites for the location of the load test. Vroman, et al concluded that the most critical site, with respect to potential failure under elevated canal water levels during the load test, was identified between GDM Stations 107+00 to 114+00 on the east levee. This site was designated by the Vroman, et al. as Site #1. The test section under consideration for the proposed load test is centered approximately on GDM Station 108+75 within this reach. In their report, the authors concluded the following:

> *"There is some reason for concern at Site #1 because of low blow counts in the sand. The controlling failure mechanism at site may be pore pressure induced instability, as opposed to seepage and piping. Therefore, there is little to no visual warning signs for this failure mechanism.*"

In their report, Vroman, et al recommend additional field investigation and testing to further evaluate the stratigraphy and soil properties at the selected test site.

5.2 CONCLUSION AND RECOMMENDATION

Review of the site selection report and supporting test data suggests that Site #1 is likely the most critical site along London Avenue Canal with respect to potential failure under elevated canal water levels during the test. The authors concur with this recommendation regarding location of the proposed test site.

5.3 CANAL SAFE WATER ELEVATION

The present maximum SWE in the canal was established by ERDC at El +4 NAVD. During a June 11, 2007 conference call with USACE [(6)], Dr. Tom Brandon of Virginia Tech (a consultant to USACE on this project) indicated that this SWE was set following issuance of the Final Draft IPET Report. The El +4 NAVD level was established based on concerns raised about the possibility that permanent deformation of the I–wall may have occurred during the Hurricane Katrina high water level of about El +8 NAVD and that a gap currently may exist between the I-wall and levee clay embankment along some portions of the I-wall. By maintaining the maximum water level in the canal at El +4 NAVD, it was believed that water would not reach this potential gap since it was believed that the crest of the levee was designed to be El +4 NAVD. The level grade at the north breach was reported [(1)] to be EL +4.4 NAVD. Figure 2 [(3)] of Appendix B indicates the design crest of the levee was El +4 NGVD or El +2.5 NAVD. Recent survey data presented above indicates the present crest grade on the canal side of the levee at the test section varies from about El +2.4 to + 2.6 NAVD. These results indicate that when the water in the canal is at the present SWE of El +4 NAVD, there will be water in any preexisting gaps between the I-wall and the levee embankment.

5.4 CONCLUSION AND RECOMMENDATION

The present SWE of El +4 NAVD appears to have been based on the assumption that the crest of the levees along the London Avenue Canal was at least at this grade. Now that it is known that the actual crest grades may be below this grade, the test will help estimate whether El +4 NAVD is, in fact a SWE. The authors believe that USACE should determine if any other levee sections are below El +4. The action needed if they are below this level may be to raise the grades to conform to this assumed level. The authors understand that USACE is currently reviewing recent survey data to verify the elevation of the crest of the levee and will provide summary findings and recommendations to parties responsible for assigning the SWE.

6. ALTERNATIVE FAILURE MODE

Prior to performing the load test, questions were raised within USACE as to the possibility of an alternative mode of failure (in addition to the gap formation and migration). This alternative mode was suggested because sand was found to exist in the bottom of the canal just outboard of the proposed load test cofferdam location in one of the geoprobe borings described above. In prior investigations of the failure mechanism, it had been assumed that marsh clay or silt covering the bottom of the canal prevented direct hydraulic connection between the canal water and the underlying sand. The impact of sand in the bottom of the canal was evaluated in several USACE reports discussed below

6.1 SEEPAGE AND STABILITY ANALYSES

Brandon [(7)] evaluated exit gradients near the protected side toe of the levee and estimated the stability of the test section with the assumption that sand was in the bottom of the canal and the water level in the canal was at El +6 NAVD. Brandon's study found that the I-wall-levee would likely fail due to high exit gradients. In a separate analysis he also estimated uplift pressures on the base of the marsh clay of Stratum B due to water in the cofferdam at El +6 NAVD under two cases [(8)]. In this analysis, the initially proposed cofferdam location on the in-canal slope of the levee was designated "Test Section". An expanded cofferdam, which included the provision that canal water was in direct contact with the sand, was designated "Full Canal". Figure B-16 [(8)] of Appendix B summarizes Brandon's analysis and illustrates the impact of the two cofferdam locations. Both analysis cases consider two alternatives. One alternative assumes that no gap exists between the I-wall and the levee embankment clay soils. The second assumes that a gap forms between the I-wall and the levee embankment clay soils during the load test, providing full canal hydrostatic pressure to the surface of the sands through the gap. The impact on this gap on water pressures in the sands at the base of the marsh clays was discussed above. For the Test Section case with no gap formation, the uplift pressures are less then the overburden pressures for all locations from the toe of levee on the protected side to a distance 30 feet beyond the toe. For the Full Canal case with no gap, the uplift pressures are almost three times the overburden pressures. This illustrates the impact of canal water elevation, particularly when the canal is in direct contact with the surface of the sands. When a gap was introduced into the analysis of the Test Section case, the uplift pressures were increased to about 2.5 times the overburden pressures for the Test Section case. When the gap was added to the Full Canal case, the uplift pressures were only slightly higher than for the Full Canal case with no gap.

6.2 SOIL STRUCTURE INTERACTION ANALYSIS

Schwanz [9] performed a FLAC soil-structure interaction analysis of the test section to assess the implication of the IPET findings and to evaluate the effectiveness of the potential load test. The major difference in the subsurface profile at the north breach compared to the test section is that the clay layer does not exist in the bottom of the canal through out the test section. This implies that the subsurface conditions at the test section are more critical than those at the area of the north breach. Triaxial and pressuremeter test results were used to evaluate the stiffness of the levee embankment and marsh cays. The pressuremeter test yielded a higher stiffness than the triaxial test as would be expected since the pressuremeter is an in situ test. To reflect the potentially critical subsurface conditions at the test section, the numerical model by Schwanz did not include the marsh clay layer of Stratum B in the bottom of the canal as was included in the IPET Appendix 9 [1] analysis. Similar to the previously referenced analyses by Brandon, these analyses by Schwanz considered that the bottom of the canal was in direct contact with the sand. In the Schwanz analysis, the gap reaches the top of the sand layer at canal water levels of El +5 and +6 NAVD, respectively, for the triaxial and pressuremeter stiffness. These values are compared/contrasted to the gap initiation water levels of El+6 and +8 NAVD when a clay layer existed in the bottom of the canal (i.e., the conditions used in the IPET analyses).

6.3 CONCLUSION AND RECOMMENDATION

The Schwanz analysis results indicate the most critical potential I-wall/levee failure mode occurs when the canal water has direct contact with sand in the bottom of the canal. The authors recommend that both the gap failure mode and the critical under-seepage failure mode be evaluated in the load test. As described in the following section, USACE has suggested modifications to the load test program to incorporate these recommendations.

7. MODIFICATIONS TO LOAD TESTING PROGRAM

In preparing this report, the authors considered all of the information presented in the previous sections. This section discusses proposed modifications to the I-wall/levee load test that were developed in recognition of the information summarized in the previous sections. The proposed modifications include: (i) expanding the lateral extent of the cofferdam; (ii) performing a two stage load test; (iii) increasing the depth of the sheetpiling in the cofferdam; (iv) performing additional subsurface characterization and analyses prior to the load test; (v) modifying the monitoring program to focus more on pore pressures; (vi) modifying the loading approach; and (vii) enhancing the monitoring and management of the load test.

7.1 LATERAL EXTENSION OF COFFERDAM

During a June 11, 2007 conference call with the USACE [6], it was noted that an evaluation of potential worst case conditions of canal seepage and stability should be considered for any future changes in the SWE. The worst case condition would include an assumption that the bottom of canal is in direct contact with the sand. Thus, it did not appear to the authors that the initially planned cofferdam configuration for the load test complied with this analysis assumption. An alternate cofferdam location was proposed by the authors to comply with the stated analysis assumption during the conference call. The suggestion was to expand the cofferdam by extending it to the middle of the canal (i.e., about 60 feet from the I-wall), so as to include the sands in the test section. Geoprobe Boring S1LCGP-3, located within this revised cofferdam area, indicated that sand existed at the base of the canal. In discussions with USACE regarding the design considerations for the load test program, the authors understand that USACE recognized that if the cofferdam was located in the center of the canal to facilitate this critical analysis conditions, the pumping schedule for the New Orleans Sewer and Water Board (NOSWB) would be adversely impacted and the risk for interior flooding would increase. For these reasons, USACE selected the cofferdam location at the base of the levee

During a second conference call on June 13, 2007 [(10)], USACE proposed an alternative cofferdam configuration. In this case, the sheetpile wall for the cofferdam was to be located 35 feet from the I-wall or about 0 to 5 feet beyond the base of the in-canal side toe of the levee. To comply with the USACE assumption that the base of the canal penetrates the sands, it was proposed by USACE that vertical infiltration pipes be installed through any marsh clay and into the sands within the cofferdam area beyond the canal side toe of levee. The authors suggested that calculations should be completed to evaluate the depth and spacing of these pipes such that the new alternate configuration would model the condition in which the cofferdam extend to the center of the canal

The requested calculations were completed by Vroman and Brandon [(11)] on June 15, 2007 in a report titled *Analysis of Injection Wells for London Avenue Canal Test Section.* The analysis results found that when the cofferdam was located 5 ft from the toe of the levee, the required tip grade for injection wells was El -31.5 NAVD, the tip elevation of the partially penetrating sheetpiling. The required injection well penetration into the sand is thus about 22 feet. The analysis results also indicated that 6-in.diameter wells will be required with a 5 ft center-to-center spacing to provide the required pore pressures equivalent to the Full Canal case discussed above. A flow rate of about 25 cfm will be required if water is at El +8 NAVD in the cofferdam.

The report by Vroman and Brandon [(11)] provides the basis for use of the proposed cofferdam at a distance of 35 feet from the I-wall with an injection well system for the load test in lieu of moving the cofferdam to the center of the canal. This alternative has the advantage that it conforms to the Full Canal analysis discussed previously yet does not severely restrict flow in the canal in case of a storm event during the load test. A final advantage of moving the sheetpiles is that at the conclusion of the load test, USACE recommends pulling the sheetpiles. Pulling sheetpiles that are driven into sand will be less concern than pulling sheetpiles that are advanced through the levee. In the case of sands, the void left by the extracted sheetpile will be rapidly closed. In the case of a sheetpile through clay (i.e., pulling sheetpile advanced through the levee), provisions need to be included to assure that the void left by the extracted sheetpile is sealed so as to minimize the potential of compromising the integrity of the levee. Based on the preliminary recommendations by the authors and the subsequent analyses presented by Vroman and Brandon[11], the authors understand that USACE has modified the original load test program to fully incorporate the recommendations provided in this section regarding the location of the cofferdam and the elevation of the injection pipes.

7.2 CONCLUSION AND RECOMMENDATION - LATERAL EXTENSION OF COFFERDAM

The authors believe that the revised load test program proposed by USACE that incorporates these recommendations is acceptable and will meet the USACE analysis condition for a worst-case analysis consideration of sand in direct contact with water at the bottom of the canal. The referenced analysis results illustrate that the gap does not have as significant an impact on the uplift pressures for the analysis case of sand in the bottom of the canal. Most importantly, however, these results indicate the impact of the direct contact between the canal water and the sand. In this latter case, under-seepage through the underlying sand controls stability of the levee on the protected side of the canal. The authors recognize that these proposed modification will likely have minimal adverse impact on the pumping schedule desired by NOSWB.

7.3 TWO STAGE I-WALL LOADING TEST

The inclusion of injection wells inside of the cofferdam led to the possibility that a two-phase loading program may be beneficial. The initial phase will consist of loading the cofferdam as described above for the original load test but with injection wells sealed such that no water will have direct contact with the sand. This phase will test the gap development mechanism. Following this test, the injection wells will be unsealed and water in the cofferdam will have direct access to the sands below the marsh clays. The load test will then be performed in an incremental fashion similar to the loading schedule developed for the first phase test. This second phase will evaluate the potential worst-case condition of sand in the bottom of the canal.

7.4 CONCLUSION AND RECOMMENDATION - TWO STAGE I-WALL LOADING TEST

The authors recommend that USACE perform a two phase test as described above to allow evaluation of the following two failure modes: (i) the gap formation mode with canal water penetrating to the sands below the marsh clay along the interface of the levee embankment clay/marsh clay and the sheeting; and (ii) the under-seepage mode with canal water flowing directly from the canal bottom through the sand around the I-wall sheeting. The authors understand that USACE has modified the load test program to incorporate these recommendations and will incorporate injection wells in the underlying sands to simulate the effects of underseepage.

7.5 INCREASED DEPTH OF SHEETPILES

The calculations by Vroman and Brandon [(11)] indicated that the required depth of penetration of the injection wells is significantly impacted by the penetration of the sheetpiling. The cross sections for the fully and partially penetrating cases analyzed are included in Figures B-17 and B-18 [(11)], respectively, of Appendix B. Figures B-19 through B-21 of Appendix B illustrate the required depth of injection well penetration for the case of fully penetrating sheetpiling. Fully penetrating sheetpiling will extend into the bay/sound/estuarine clay. The elevation of the tip of the injection well for this case is El -22 NAVD or 13 feet into the sand. When partially penetrating sheeting was considered using sheetpile tips at El -31.5 NAVD, the required tip of injection wells was found to be at El -31.5, the tip elevation of the sheetpiling as noted above. Figures B-22 and B-23 illustrate the comparison of injection well pore pressure at the I-wall sheeting tip and the pore pressures at the protected side toe of the levee embankment at the interface between the marsh clays and the sands. The basis of comparison is the calculated pore water pressures considering the Full Canal loading scenario. These figures indicate that the partially penetrating sheetpiling can be used, but that the use of partially penetrating sheetpiles adds an additional unknown to the analysis of the load test results. The analysis by Vroman and Brandon assumes subsurface conditions that may not be the same as found below the test section. The use of fully penetrating sheeting will eliminate this unknown and help assure that the potential worst-case analysis conditions are modeled.

7.6 CONCLUSION AND RECOMMENDATION - INCREASED DEPTH OF SHEETPILES

The partially penetrating sheeting requires deeper injection wells and greater injection water volumes to simulate the condition of fully penetrating sheeting because some of the water from the cofferdam will flow beneath the sheetpiles and towards the canal instead of towards the protected side. To reduce this unknown in the analysis of the test results, the authors recommend that fully penetrating sheeting be used to construct the cofferdam. The authors understand that USACE has modified the load test program to incorporate these recommendations and will incorporate the Vroman and Brandon [11] calculation results regarding the location and depth of the sheetpiles and the depth of the injection wells.

7.7 ADDITIONAL SUBSURFACE CHARACTERIZATION AND ANALYSES

The subsurface investigation for the test section has been limited to geoprobe borings. Additional test borings penetrating the levee embankment clays and marsh clays are needed to improve the characterization the stratigraphy and to obtain samples that will be used for testing to supplement the available soil properties. In situ testing should also be considered. The available soil property data are summarized in Table B-1 of Appendix B. The table includes soil properties developed by IPET [1] for the north breach. Brandon [7,8] performed seepage and stability analyses for the test section and Schwanz [9] performed his referenced SSI analyses for the test section using essentially these same values. Specific data considered by Brandon and Schwanz in their analyses are noted in this table. The limited data obtained specifically from the test section are designated Test Section. An analysis/recommendation of the needed soil property data is included in Table B-2 of Appendix B.

7.8 CONCLUSION AND RECOMMENDATION - ADDITIONAL SUBSURFACE CHARACTERIZATION AND ANALYSES

The analyses performed for the test section by USACE should be updated with the additional subsurface information and laboratory and in situ testing described above. Specifically, six additional borings are recommended to be advanced on the protected side of the levee. Three of these borings are at the levee crest and three are located at the toe of the levee. These borings should be drilled into the sands along the three analysis cross sections shown on Figure 11 of Appendix B. Five-inch diameter undisturbed tube samples should be obtained for horizontal and vertical triaxial testing. Both CPT and DMT should be considered for characterizing the subsurface materials at each sampling location. The authors understand that USACE is currently obtaining additional five-inch diameter undisturbed tube samples and that the laboratory testing program for these samples is currently being developed. The authors understand that they will be provided the proposed laboratory and field testing program for review when developed by USACE.

7.9 FOCUS INSTRUMENTATION ON PORE PRESSURES

An extensive instrumentation network is proposed by USACE. The instrumentation plan appears to be appropriate for monitoring the planned load test. With the additional consideration of the under-seepage mode of failure as noted by Vroman, Brandon and Schwanz [(4)], additional concerns are raised about pore pressures and their impact on stability during the load test. This concern about the potential pore pressure induced instability must be addressed in the monitoring program. Pore pressures in the sand must be estimated before the test at each monitoring well location such that threshold levels are assessed at every stage of loading. This failure mechanism is the major risk associated with the second stage of the load test program and must be very carefully evaluated before proceeding with the test. The authors believe that additional piezometers should be considered for assessing under-seepage conditions. Furthermore, the authors believe that the inclinometers, the crackmeter, and the total stress cell included in the current instrumentation plan will provide less useful information than piezometers.

7.10 CONCLUSION AND RECOMMENDATION - FOCUS INSTRUMENTATION ON PORE PRESSURES

Additional piezometers should be specified. Piezometers should be placed on the slope of the levee embankment from the I-wall to the toe of the levee. These piezometers will be critical to the monitoring of the advancement of the increased pore pressures during the each load test increment. Piezometers can be used to assess the flow regime in the critical direction perpendicular to the levee, as well as to assess the potential for three-dimensional effects induced by the proposed cofferdam. Suggested piezometer locations should be developed by USACE using input from Vroman, Brandon and Schwanz prior to initiating the load test. The authors would be pleased to review and comment on the proposed instrument locations when they are selected.

7.11 MODIFY LOADING APPROACH

As proposed the I-wall will be loaded by adding water to the cofferdam in 6-in. vertical increments, with each increment being held 4 hours prior to adding the next 6-in. vertical increment of water. This approach does not explicitly provide for allowing the subsurface flow regime and/or the wall deformations to come to equilibrium under a specific loading increment.

7.12 CONCLUSION AND RECOMMENDATION - MODIFY LOADING APPROACH

The authors believe that each load increment should be held until equilibrium is reached in both wall deflection and subsurface pore pressure. The definition of "equilibrium" should be established by the ERDC prior to initiation of the load test program. Implementation of this recommendation will help in evaluating the time rate effects associated with the build up of pore pressures under sustained hydrostatic loading. Protocols should be established prior to the initiation of the load test to establish the performance criteria related to this equilibrium condition. The authors understand that USACE is currently assessing specific load duration controls to allow for the development of steady-state flow conditions prior to increasing the water levels in the cofferdam. Furthermore, the USACE recommendations will be presented in a *Synchronization Protocol Matrix* that will be provided to the authors for comment and concurrence when the modified control program is developed.

7.13 ENHANCE TEST MONITORING AND MANAGEMENT

During the recent June 19, 2007 meeting with USACE, the authors were provided a draft copy of a plan for managing decisions related to the proposed test titled *Synchronization Protocol Matrix*. This plan focused on information needed to be to/from specific individuals regarding: (i) initiating the test (e.g., confirmation that all instrument readings have stabilized); and (ii) terminating the load test (e.g., increase in pore pressure on protected side to a pre-selected level). This draft plan had not been advanced to the point of identifying the decisions that need to be made during the conduct of the load test.

7.14 CONCLUSION AND RECOMMENDATION - ENHANCE TEST MONITORING AND MANAGEMENT

The authors believe that a *Load Test Monitoring and Management Plan* needs to be developed for this project. In addition to the components already being considered in the draft plan, the authors believe that explicit responsibilities need to be established for individuals during the load test. USACE should develop this plan and include input from the entire team and provide a copy to the reviewers for comment. The authors believe that it would be a tremendous advantage to have members of the USACE and ERDC analysis team (e.g., Vroman, Schwanz, Conroy, Volkman, and Brandon) on-site during the load test and active in real time data analysis. This would likely involve the "updating" of analysis models using the actual data from the load test and then "predicting" by calculation the response of the systems (i.e., wall deflection and pore pressure response) during the next load increment. In this manner, the numerical models can be essentially calibrated with actual data and any questions can be addressed in real time prior to advancing to the next load increment. There is a unique opportunity presented by this load test to achieve this valuable feedback response and the authors recommend that this be considered in the referenced management plan. The authors understand that Messrs. Schwanz, Vroman, and Brandon will be present during the load test program. In addition, at the suggestion of the SLFPA, the authors will also be present during the load test program and will provide a summary report to the SLFPA upon completion of the load test program.

8. RECOMMENDATIONS FOR LONDON AVENUE CANAL LOAD TEST

8.1 RECOMMENDATIONS

The authors recommend that the results of studies performed by USACE and others after publication of the IPET Report be considered in the design and performance of the proposed hydrostatic load test program at the London Avenue Canal. Specifically, the authors recommend that the load test be performed to obtain information regarding the development and progression of the gap, as well as under-seepage beneath the levee.

To accomplish this, the authors recommend performing the load test in two stages. The first stage of the load test would be performed to evaluate the development and propagation of the gap, while the second stage would be focused on assessing the potential for under-seepage beneath the levee. The authors concur with load test configuration modifications proposed by USACE and agree that these modifications will facilitate the evaluation of the second stage of testing. The proposed USACE modification extends the width of the cofferdam by only 10 ft into the canal and includes installing infiltration pipes into the underlying poorly graded sand layer beneath the bottom of the canal. It is also recommended that that the depth of the sheetpiles used for construction of the cofferdam be extended into the clays underlying the sand stratum to facilitate data evaluation during the load test. In this report, the authors recommend that several additions/modifications be made to the original load test program prior to commencement of the program. These changes include obtaining additional strength and stiffness characterization data in the subsurface soils on the landside of the levee, as this information will help in the analysis of the results from the load test program, as well as assisting future activities related to extending the results of the load test program to other reaches of the levee. It is recommended that the data collection activities during the load test focus on monitoring pore pressures in the underlying sands in lieu of monitoring physical deformations of the I-wall gap, as the increase in water pressure is likely the most important end result of wall deformations. During load testing, the authors recommend that the originally proposed load testing protocols be adjusted to allow stabilization of the pore water pressure and wall deflection readings within pre-determined limits prior to incrementally increasing the hydrostatic loads. This recommendation will also allow assessing the time-dependent nature of pore pressure and wall deformations during sustained loading. Finally, the authors recognize a significant benefit in having the field-collected monitoring data collected and analyzed in real time during the load test as originally proposed by USACE. In addition, the authors believe that it would be beneficial to have individuals from USACE, ERDC, and Virginia Tech who were responsible for the post-Katrina London Avenue Canal numerical analyses maintain an on-site presence during the performance of the load test so that their analysis models can be updated in real-time using the actual performance monitoring test results. Based on verbal presentation and interactions with USACE, the authors understand that many of these recommendations will be (or have been) incorporated into proposed modifications to the load test program. Verification that recommendations were incorporated into the load test program will be included in a report prepared by the authors after execution of the load test program. In addition, this subsequent report will also identify other modifications to the load test program that are incorporated to improve the execution and the quality of information from the load testing of the London Avenue Canal.

The SLFPA requested that the technical review also include comments regarding the extension of the load test results to the evaluation of the SWE in the entire London Avenue Canal. The authors recommend that results of the two-stage hydrostatic load test be coupled with additional subsurface characterization tests performed along the London Avenue Canal and pore water pressure monitoring results along select portions of the canal during service operations be used to assess the reasonableness of any proposed increase in the SWE in the London Avenue Canal. Although it is premature at this time to identify a specific framework for assessing the results of the load test and the additional subsurface characterization testing and pore pressure monitoring, the authors envision several potential outcomes, including: (i) the additional subsurface characterization testing will help identify conditions along the canal and whether they are similar or different than the conditions in the vicinity of the load test and identify locations for installing additional instruments (e.g., piezometers)); and (ii) installed piezometers along the canal can be monitored during service conditions and their responses compared to the piezometer response during the load test such that the load test results can be used to help estimate the response time for underseepage under increased water elevations, possibly allowing the duration of loading to be used to control the SWE. Preliminary recommendation regarding how the results of the load test program can be extended to help establish the SWE will be presented in the referenced report by the authors at the completion of the load testing program.

APPENDIX A

REFERENCES

1. Interagency Performance Evaluation Task Force, Final Draft Report, Volume V, June 1, 2006.

2. General Design Memorandum 19A, London Avenue Outfall Canal, Orleans Parish, LA, U.S. Army Corps of Engineers, New Orleans District, New Orleans, LA, Volume 1 and 2, January 1989.

3. Site Specific Load Test Information and Addendum CD's provided Corps of Engineers, May and June 2007.

4. London Avenue Canal Load Test – Small Load Test Site Selection, Noah Vroman (ERDC), Thomas Brandon (Virginia Tech), and Neil Schwanz (MVP), undated.

5. Design of Automated Data Acquisition System (ADAS), URS, May 2, 2007.

6. Conference Call between Reviewers and Corps of Engineers New Orleans District, ERDC and Virginia Tech Personal, June 11, 2007.

7. Analysis of the London Avenue Canal Load Test - Section 1, Seepage and Stability Analysis, Thomas L. Brandon, February 10, 2007

8. Results of Seepage and Stability Analyses on Test Section Geometry, E-mail from Tom Brandon to Noah Vroman and Pat Conroy, May 5, 2007.

9. Analysis of the London Avenue Canal Load Test - Section 1, Neil Schwanz (MVP), February 21, 2007.

10. Conference Call between Reviewers and Corps of Engineers New Orleans District, ERDC and Virginia Tech Personal, June 13, 2007.

11. Analysis of Injection Wells for London Avenue Canal Test Section, Thomas Brandon and Noah Vroman, June 15, 2007.

12. Results of Under-seepage Analyses of 17th Street, Orleans Avenue, and London Avenue Outfall Canals, Pat Conroy, May, 2006.

13. Analysis of the London Avenue Canal Load Test, Impeded Drainage Analysis, Thomas L. Brandon, February 10, 2007

APPENDIX B

FIGURES AND TABLES

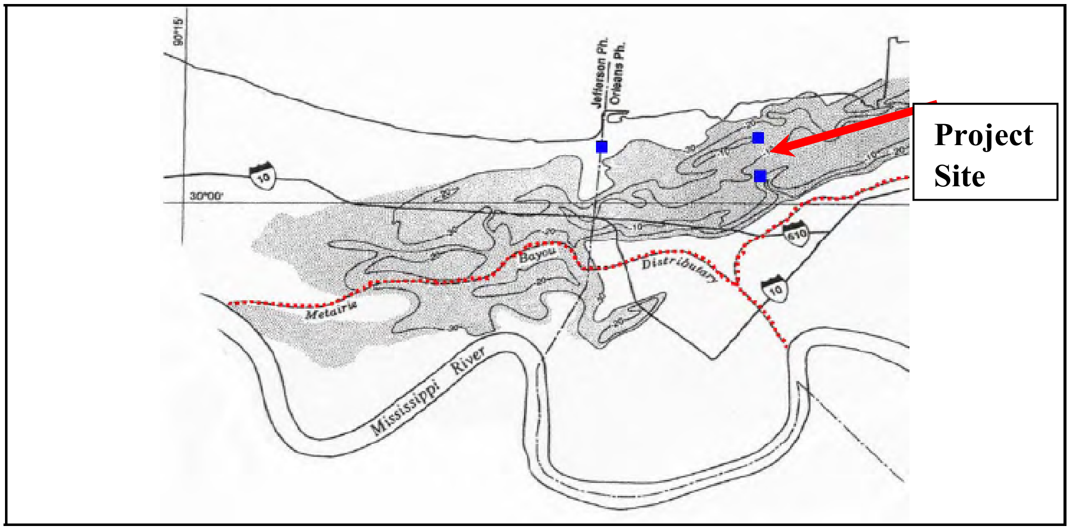


Figure 1-4b. Close-up view of the buried beach ridge, and the locations of the canal breaches to the buried beach (after Saucier 1994). The 17th Street breach is located behind the axis of the beach ridge while the London Canal breaches are located on the axis of the ridge. Bayou Metairie is identified in red and forms the Bayou Sauvage distributary course (No. 11) in Figure 1-2.

Figure B-1 –Pine Island Buried Beach Sand (from IPET Volume V, Appendix 1)

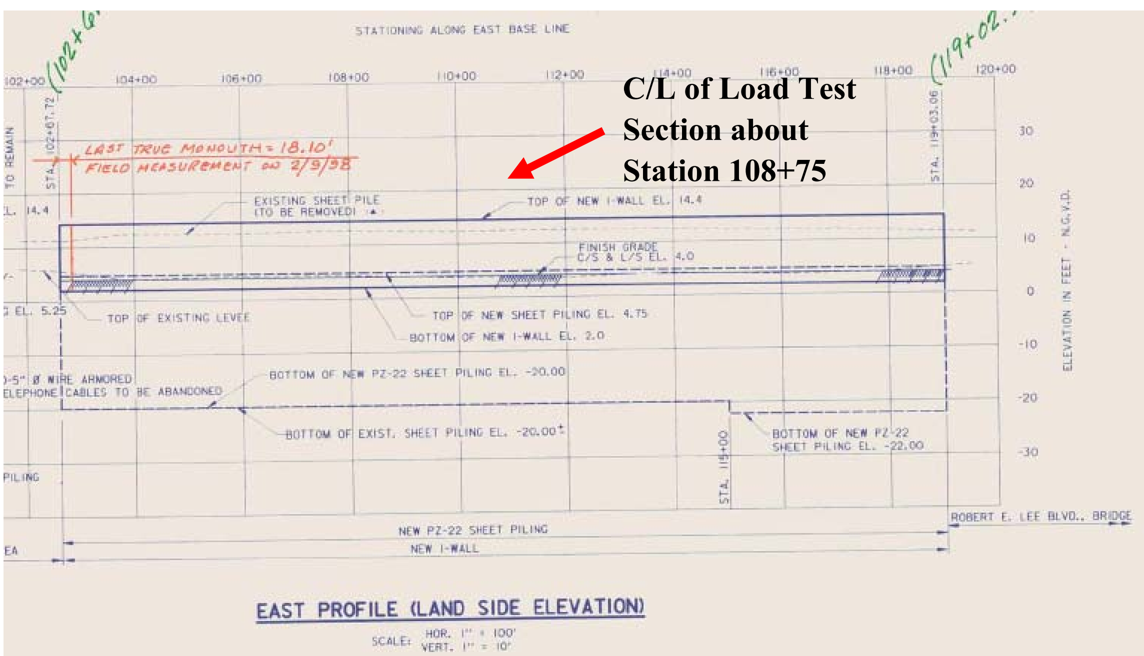


Figure B-2 – Profile along I-Wall (NGVD Datum)

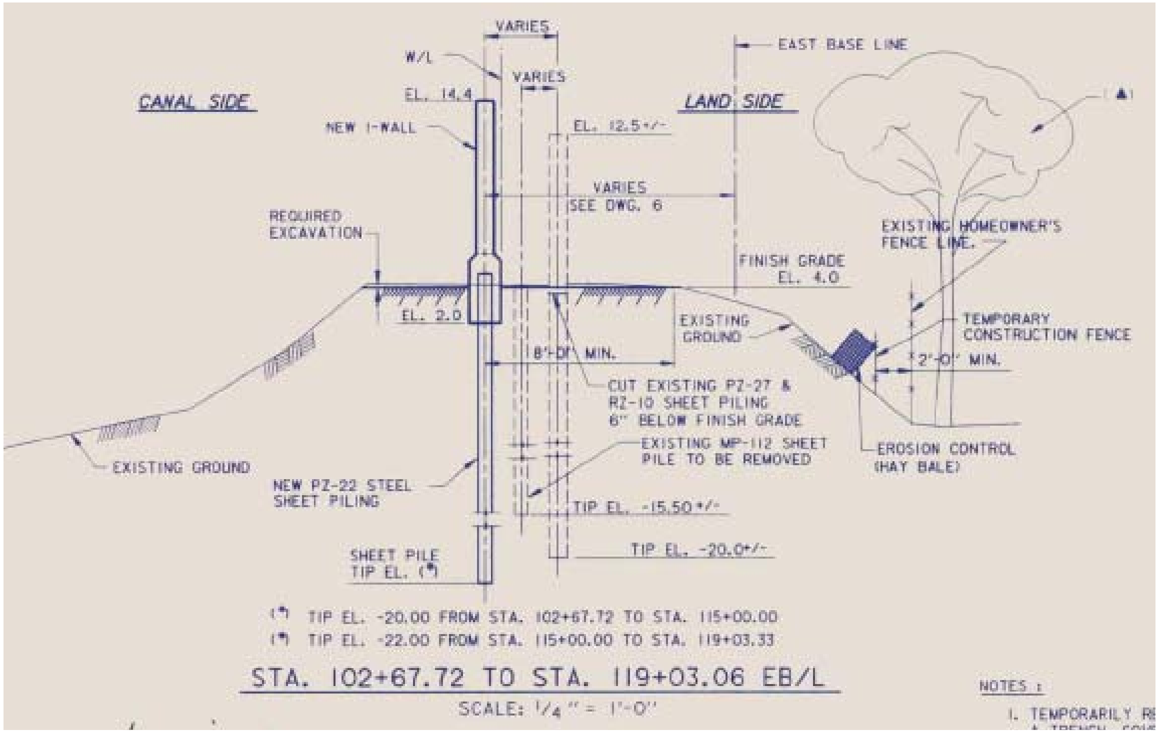


Figure B-3 – Section through I-wall (NGVD datum)

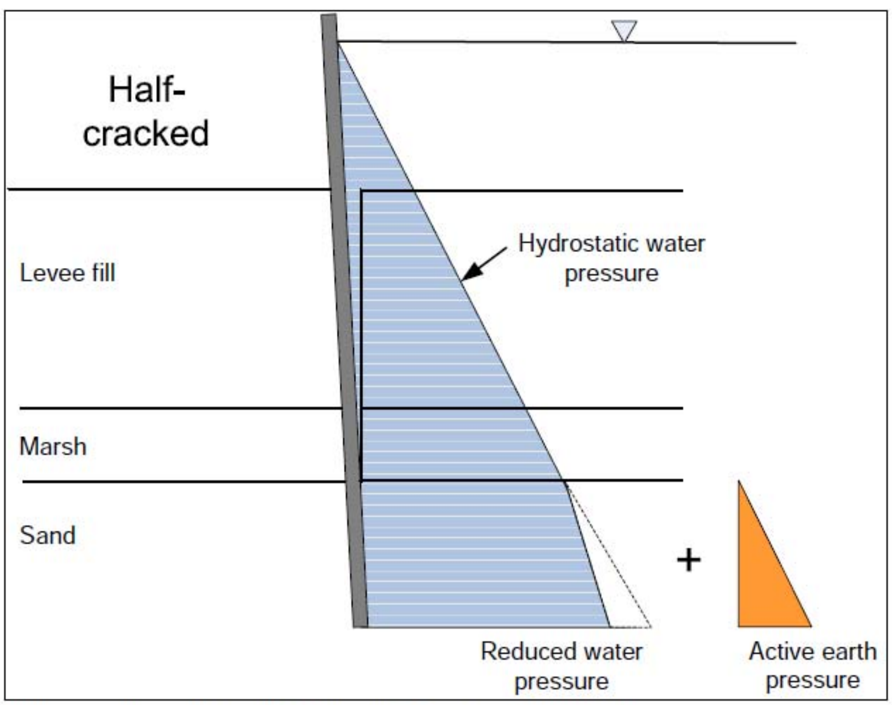


Figure 8-5. Schematic of Crack Used in London Avenue Canal Stability Analyses

Figure B-4 – Schematic Cross Section of Half Cracked Model

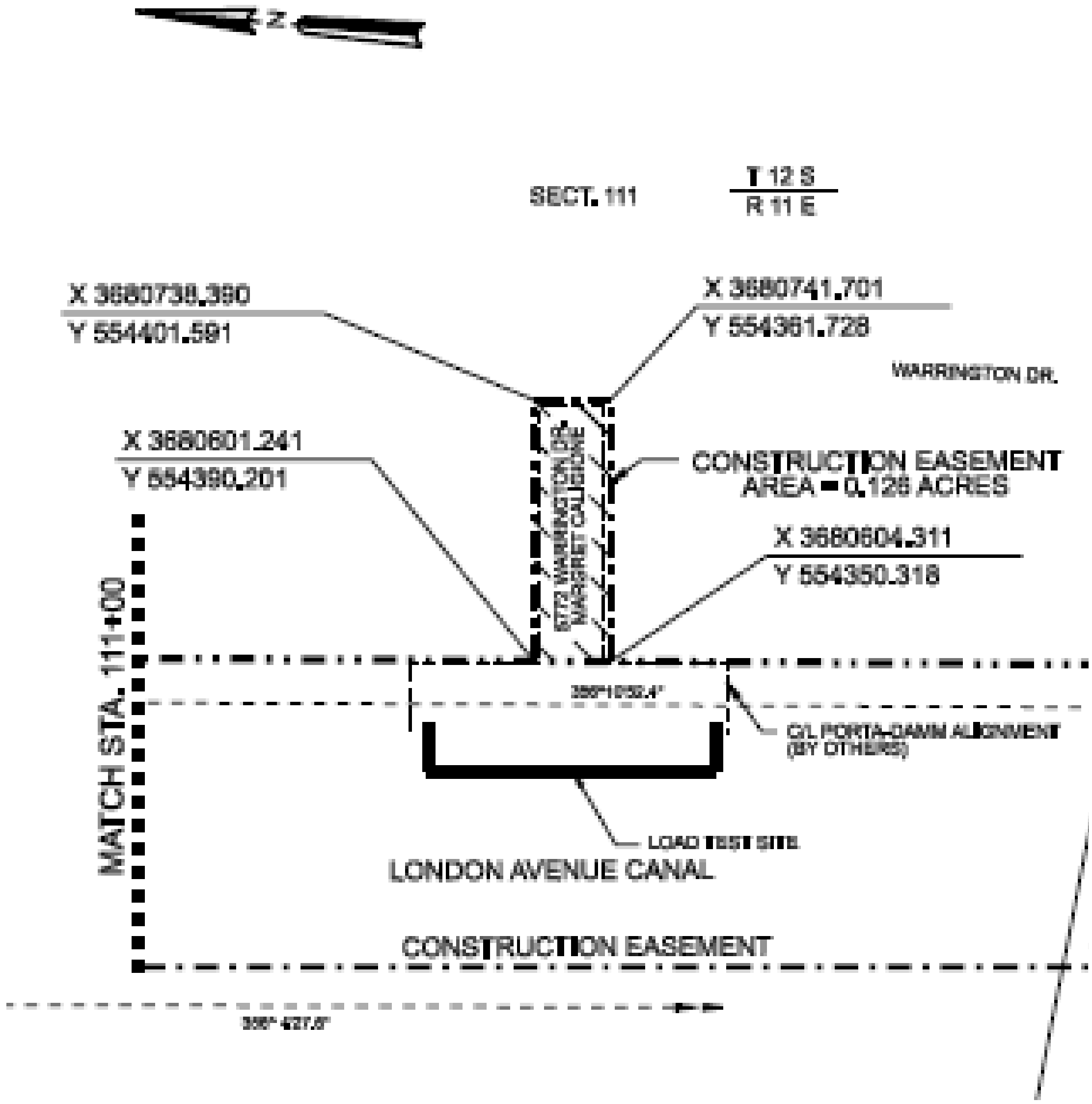


Figure B-5 – Plan View of Coffer Dam

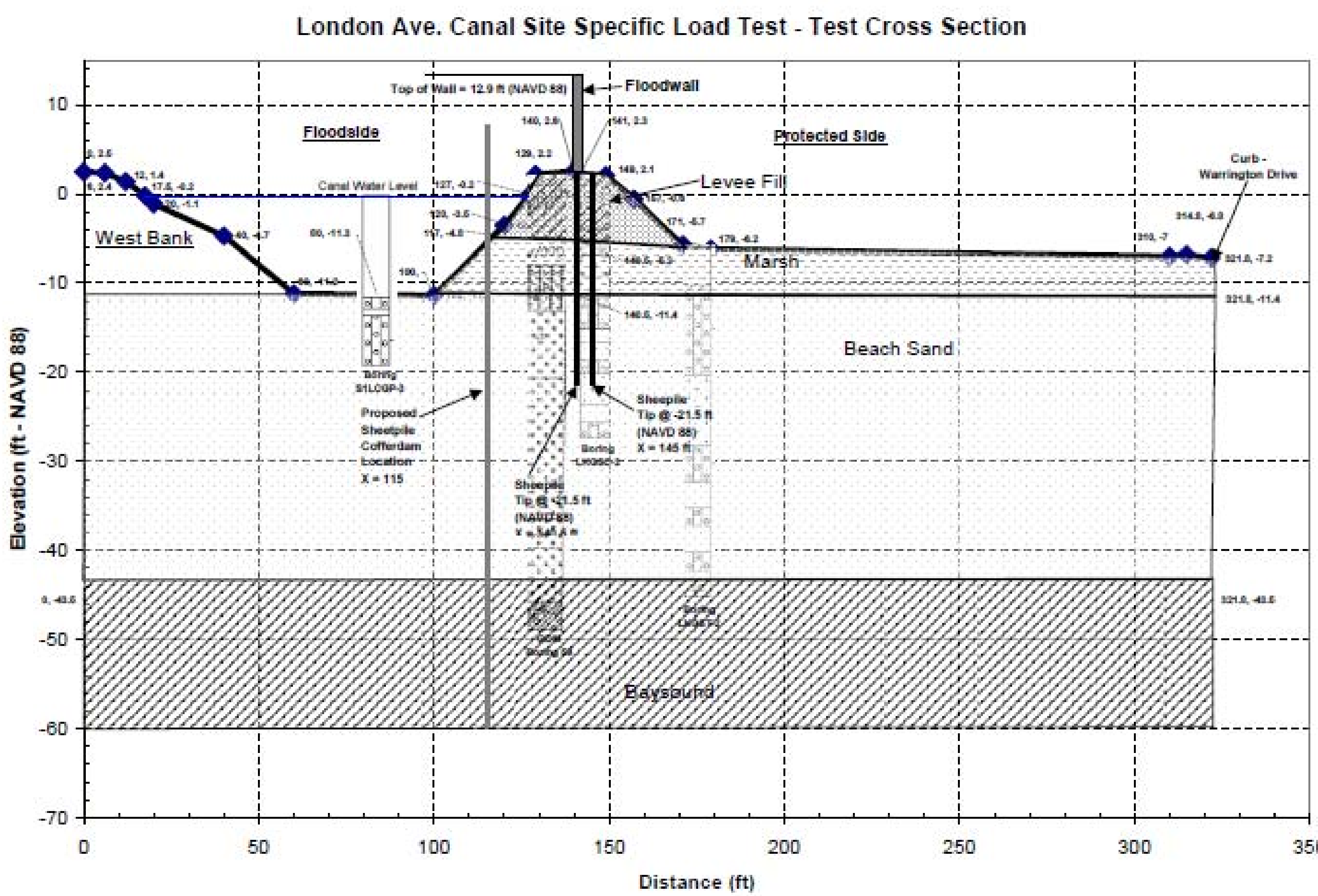


Figure B-6 – Subsurface Profile at about GDM Station 8+75 (Test Section Centerline)

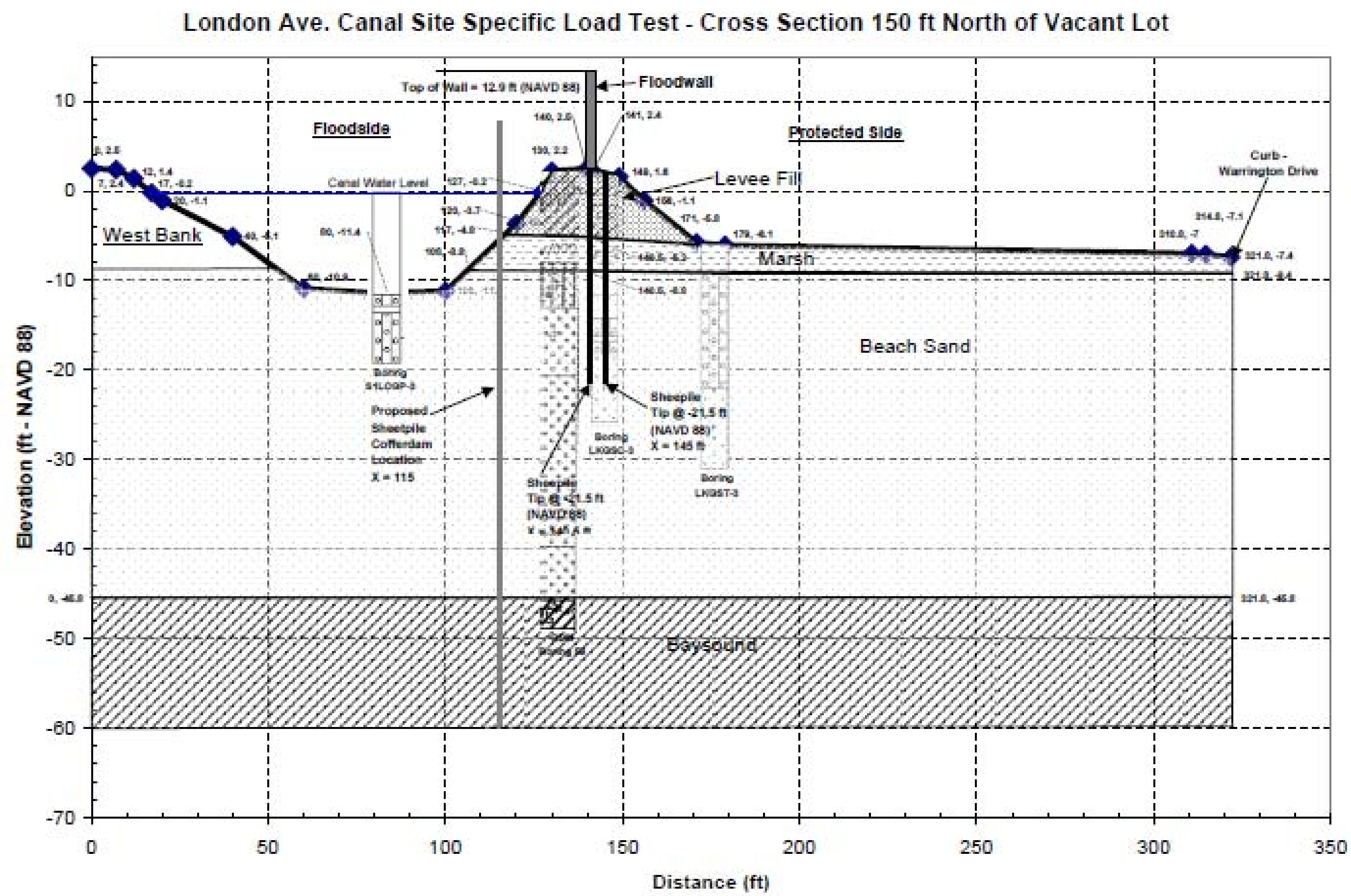


Figure B-7 – Subsurface Profile at about GDM Station 9+00

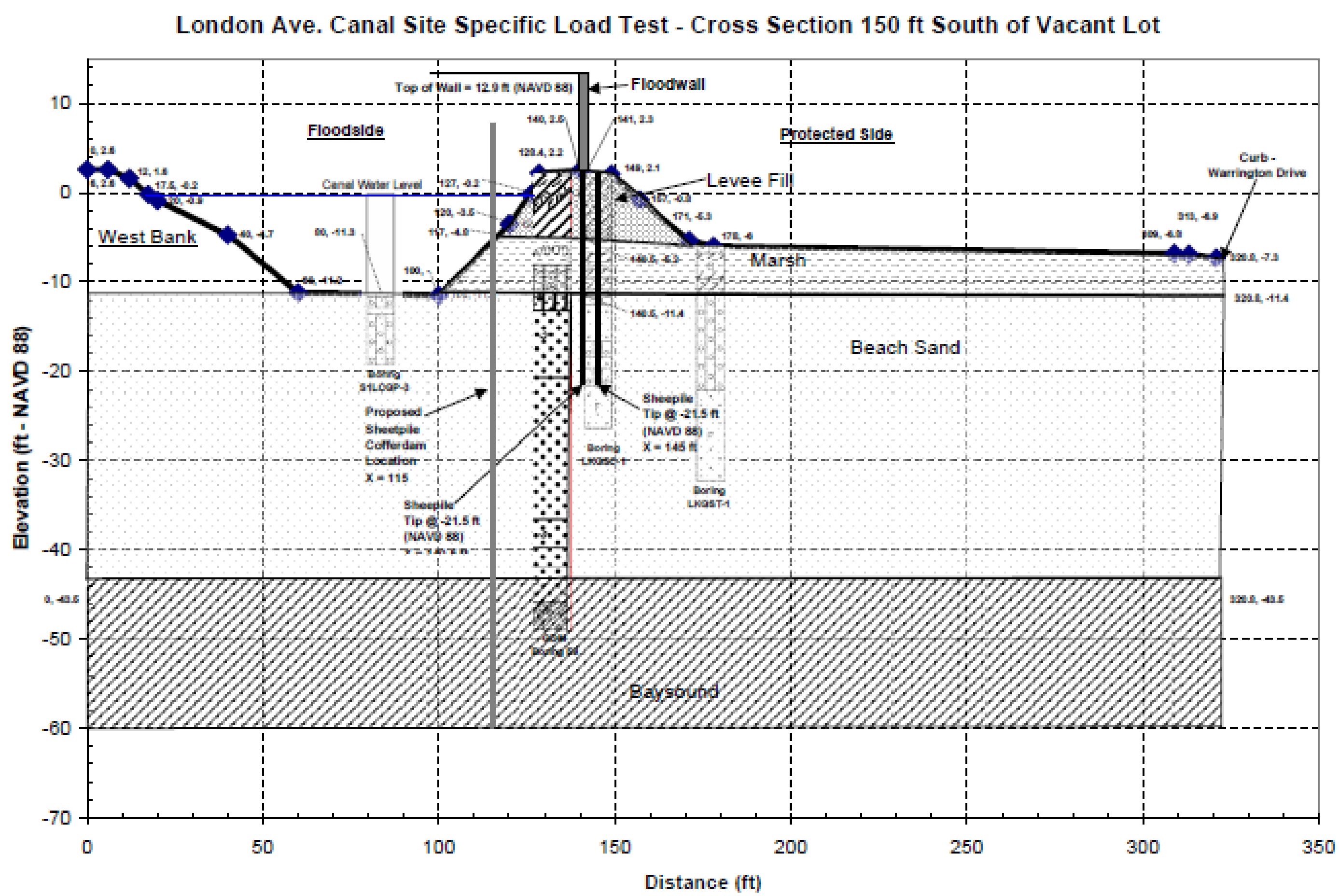


Figure B-8 – Subsurface Profile at about GDM Station 8+50

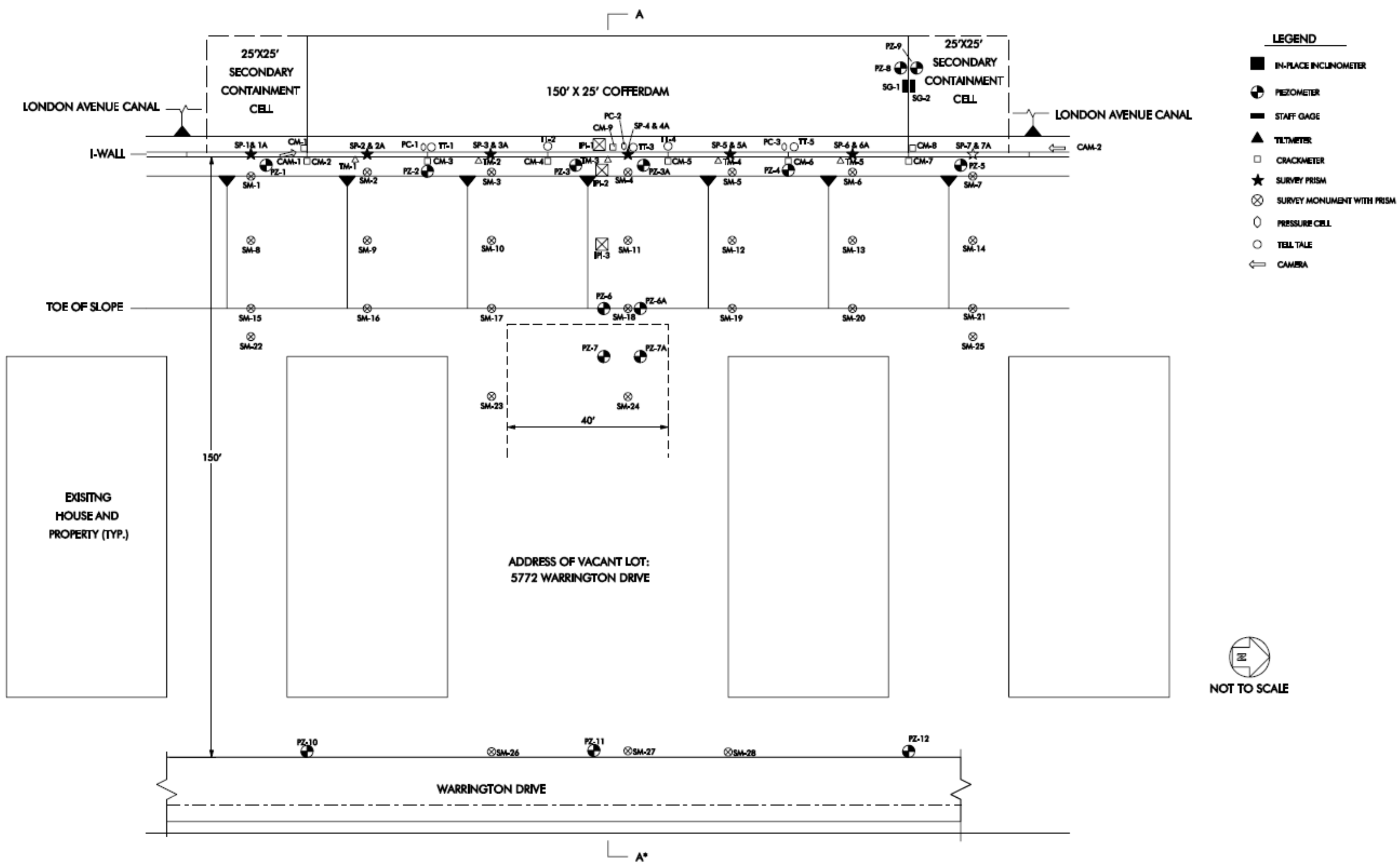


Figure B-9 – Plan of Instrumentation

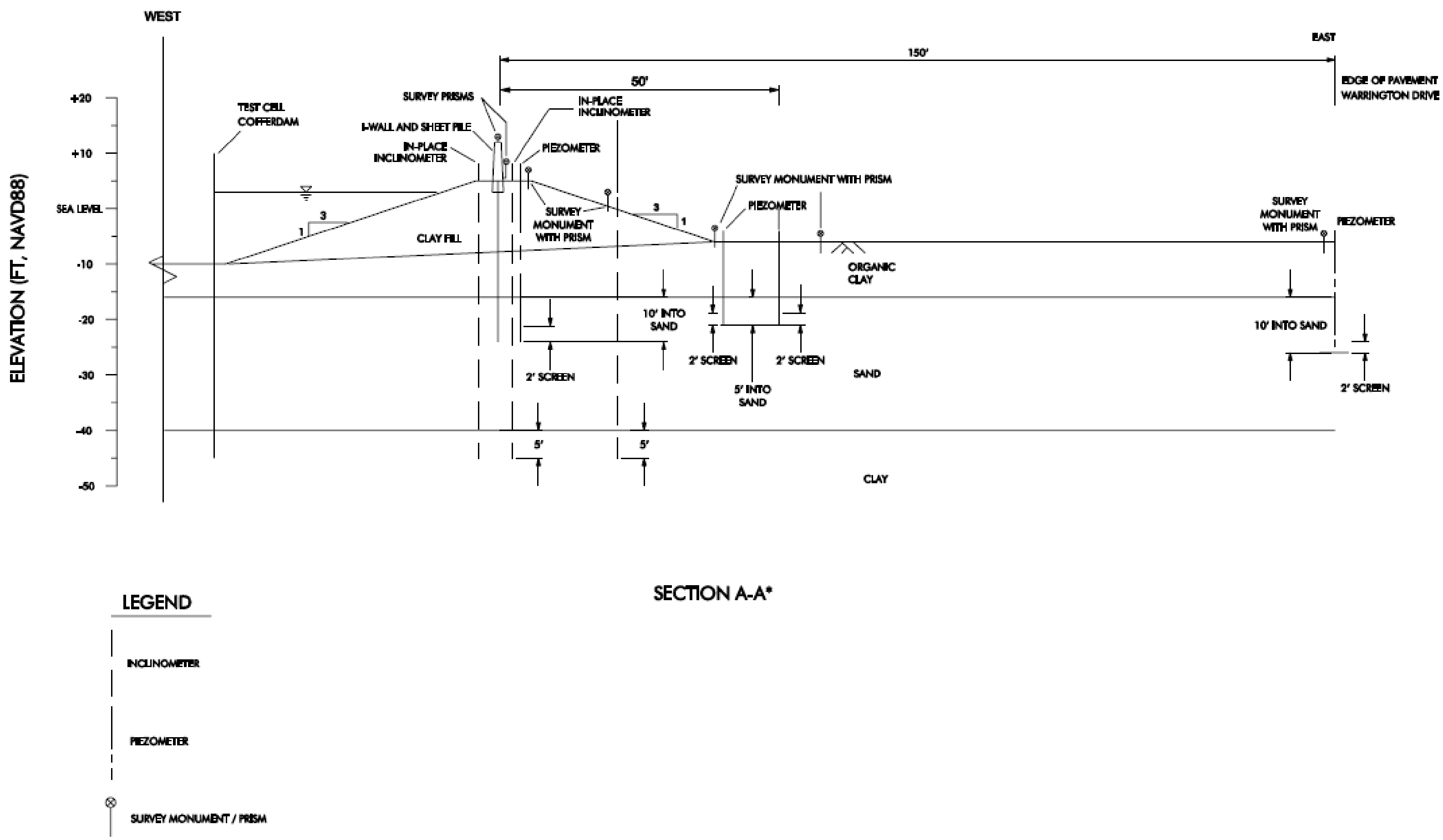


Figure B-10 – Cross Section of Instrumentation

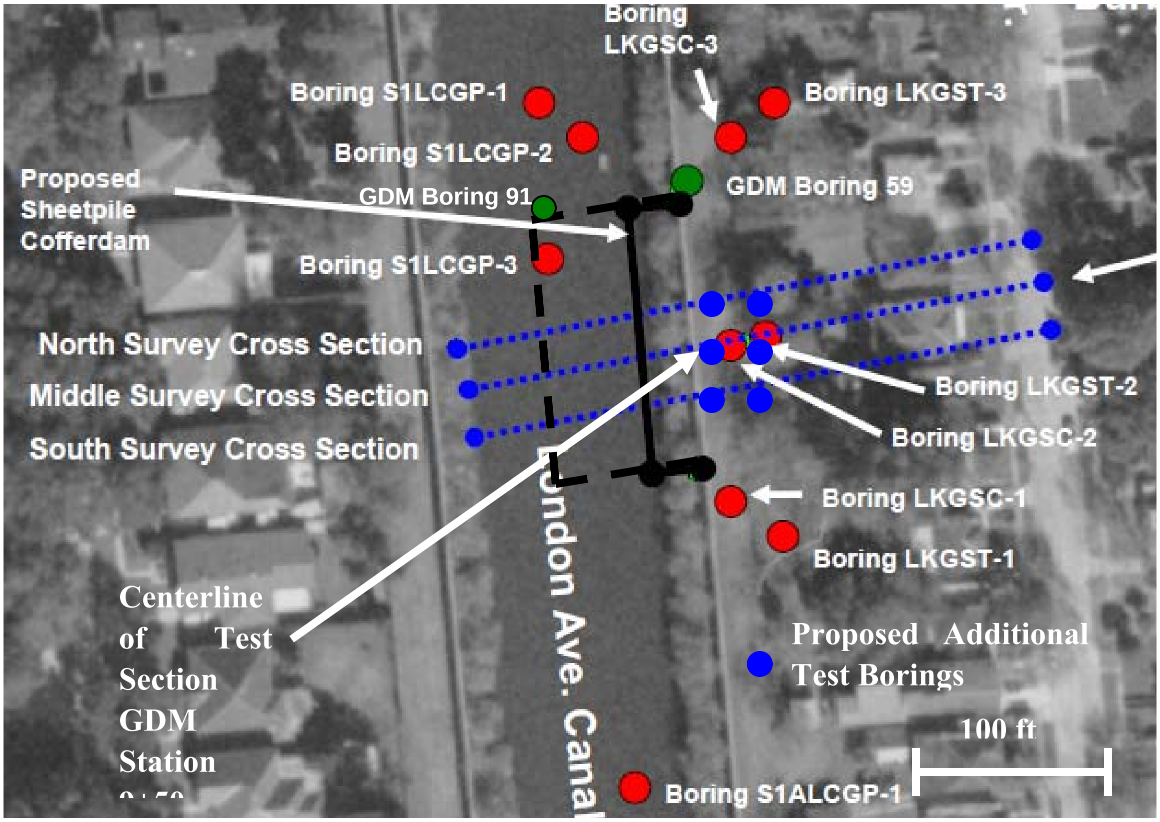


Figure B-11 – Boring Location Plan

Figure B-12 – View of I-Wall/Levee Test Area from Lee Bridge

Figure B-13 - View of I-Wall/Levee Load Test Area from Warrington Drive

Figure 31. Tilted I-Wall Opposite the North Breach on the London Avenue Canal

Figure B-14 – Deflected I–Wall North of Load Test Site

Figure B-15 – View of Protected Side Toe of Levee in Test Area Looking North

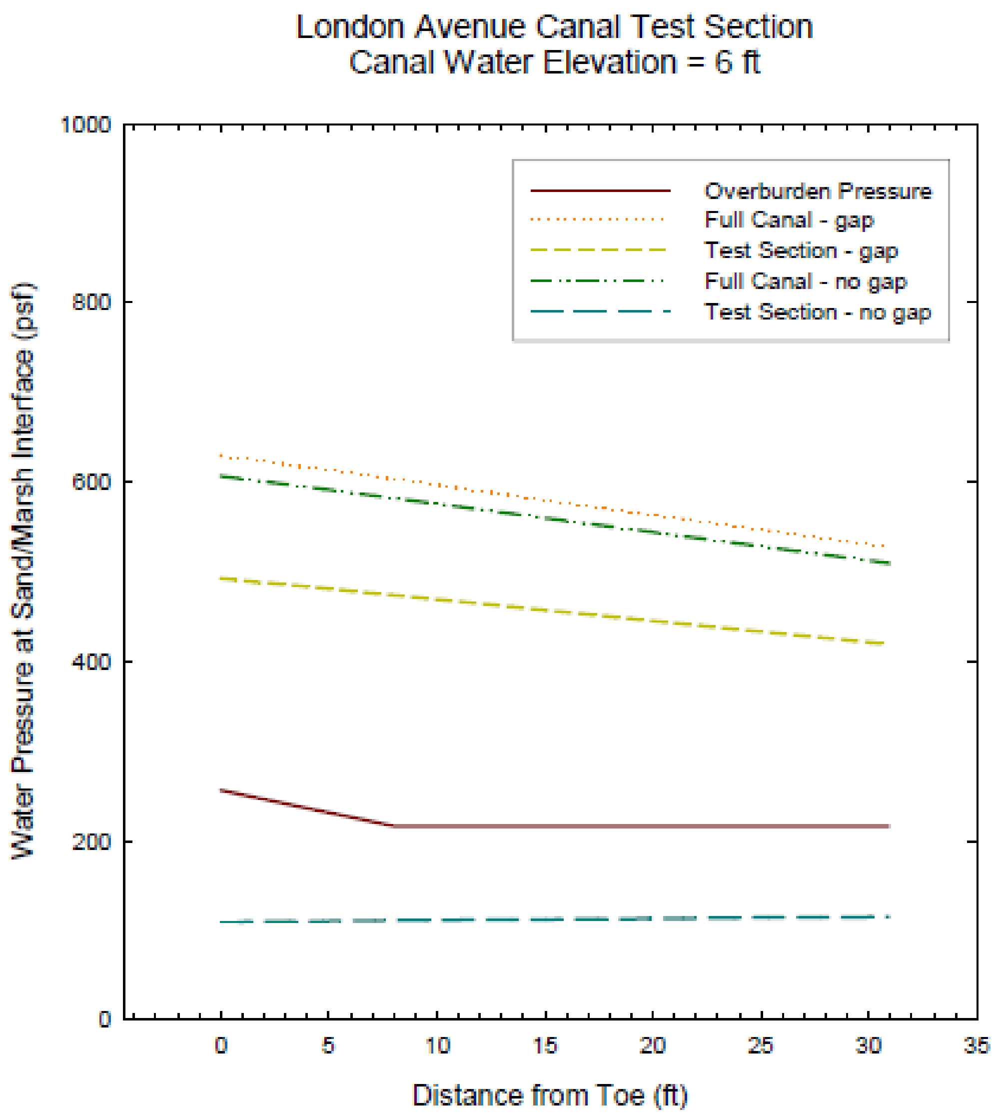


Figure B-16 – Comparison of Water Pressures at Stratum B/D Interface for Canal Sand Bottom with/without a Gap and Test Section with/without a Gap

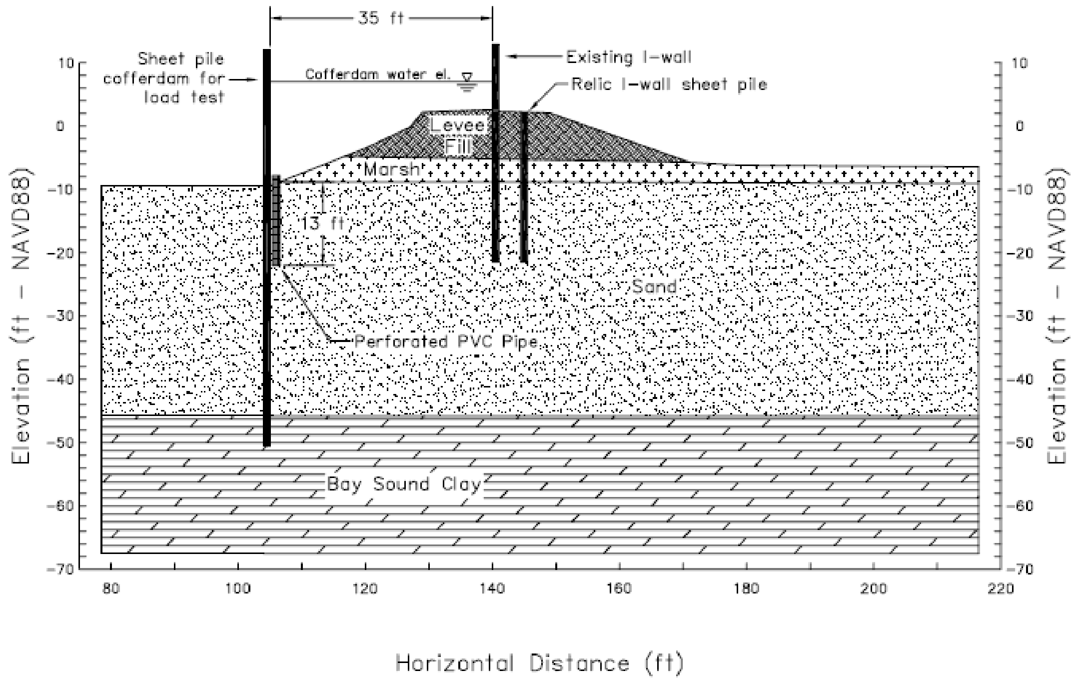


Figure B-17 – Cross Section for Analysis of Injection Wells with Fully Penetrating Sheeting

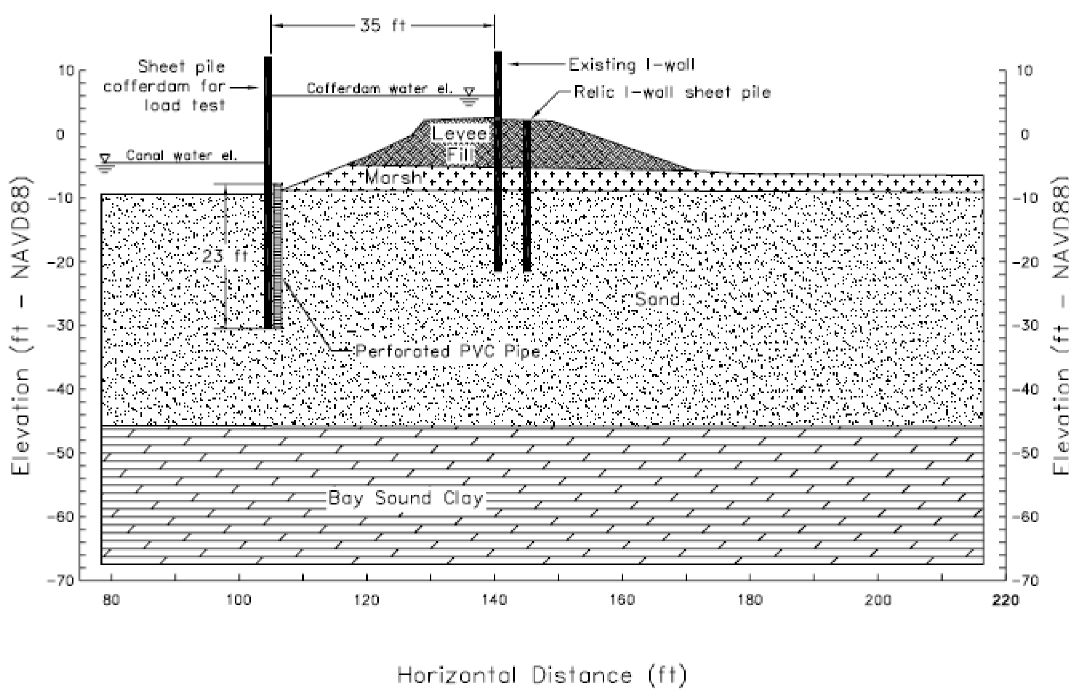
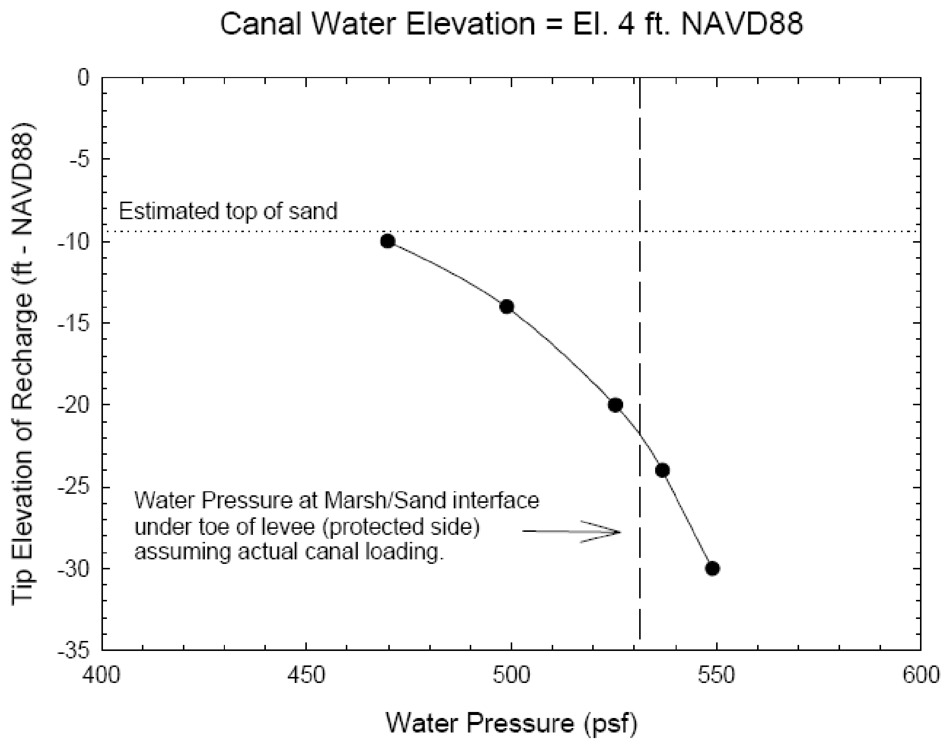


Figure B-18 – Cross Section for Analysis of Injection Wells with Partially Penetrating Sheeting



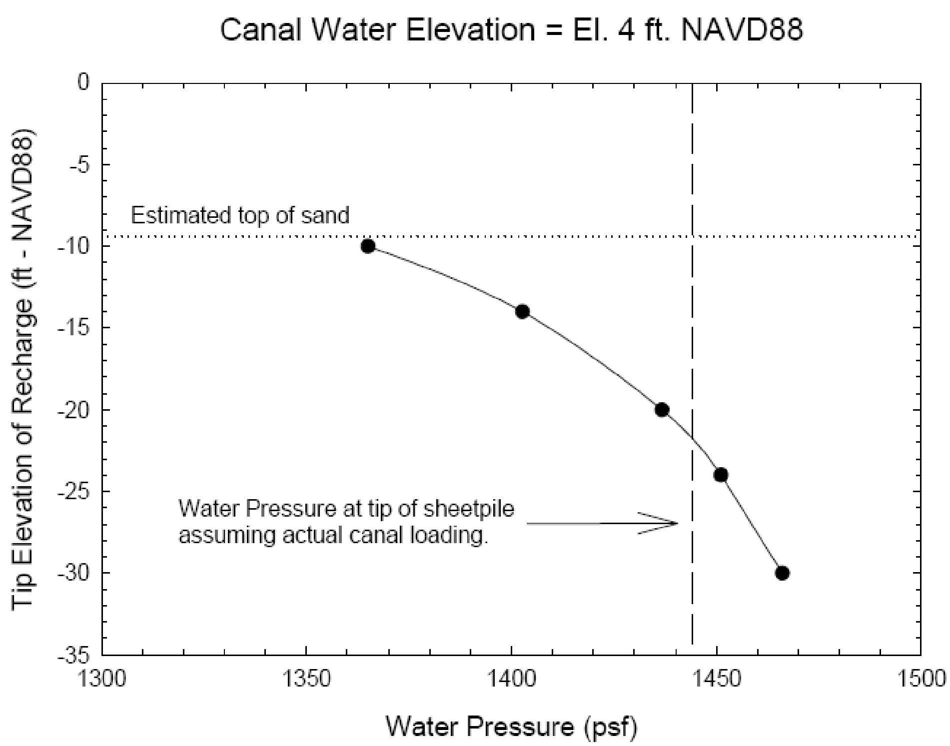
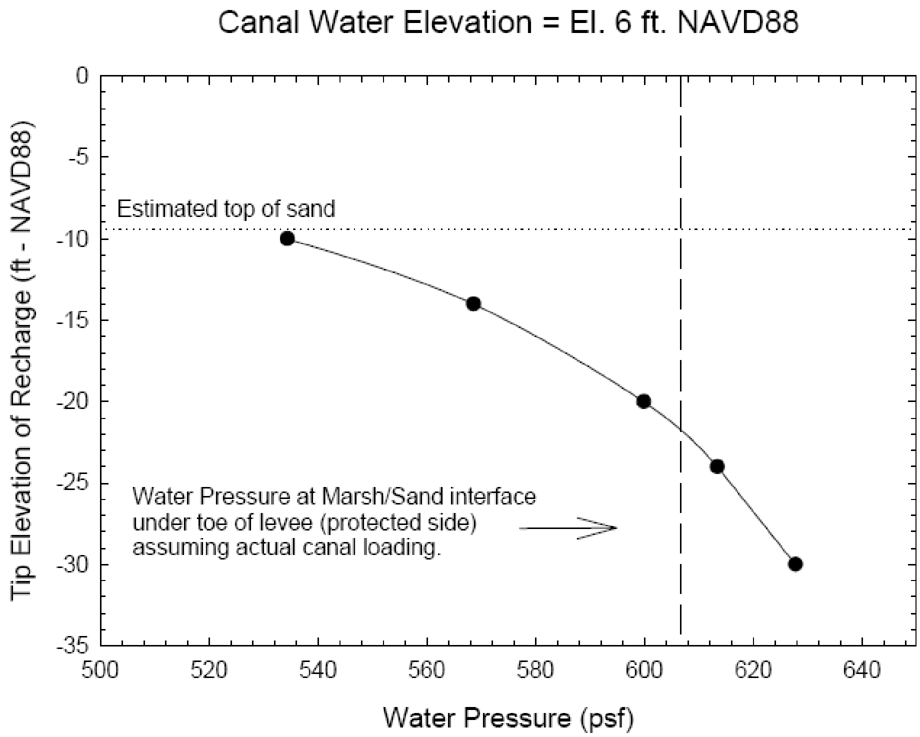


Figure B-19 – Depth of Penetration of Injection Wells for Canal Water at El +4 NAVD for Fully Penetrating Sheetpiling



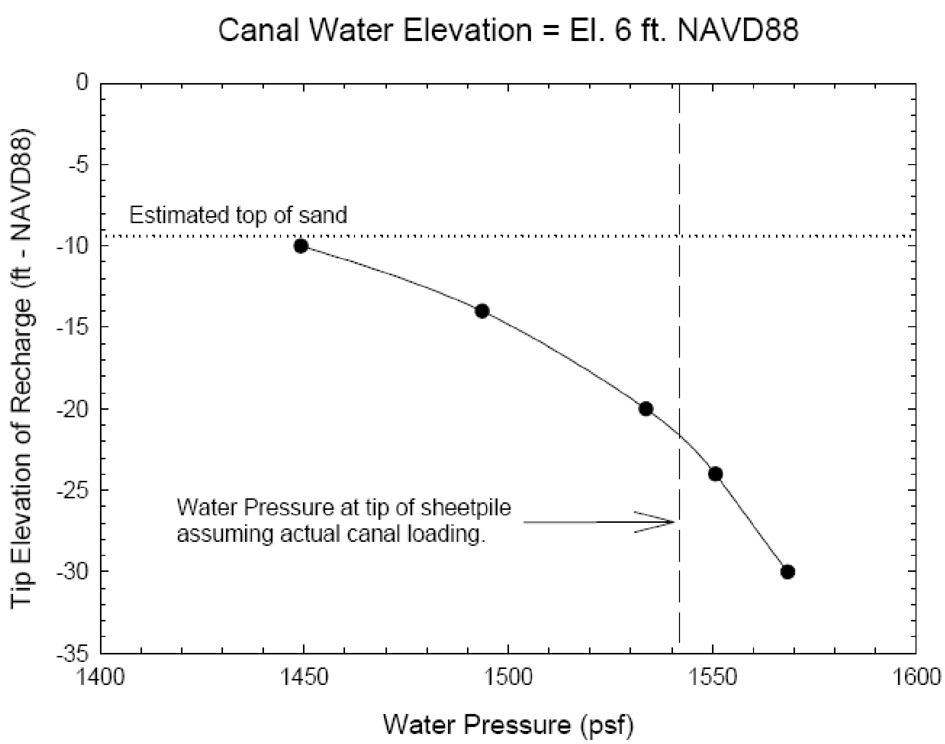
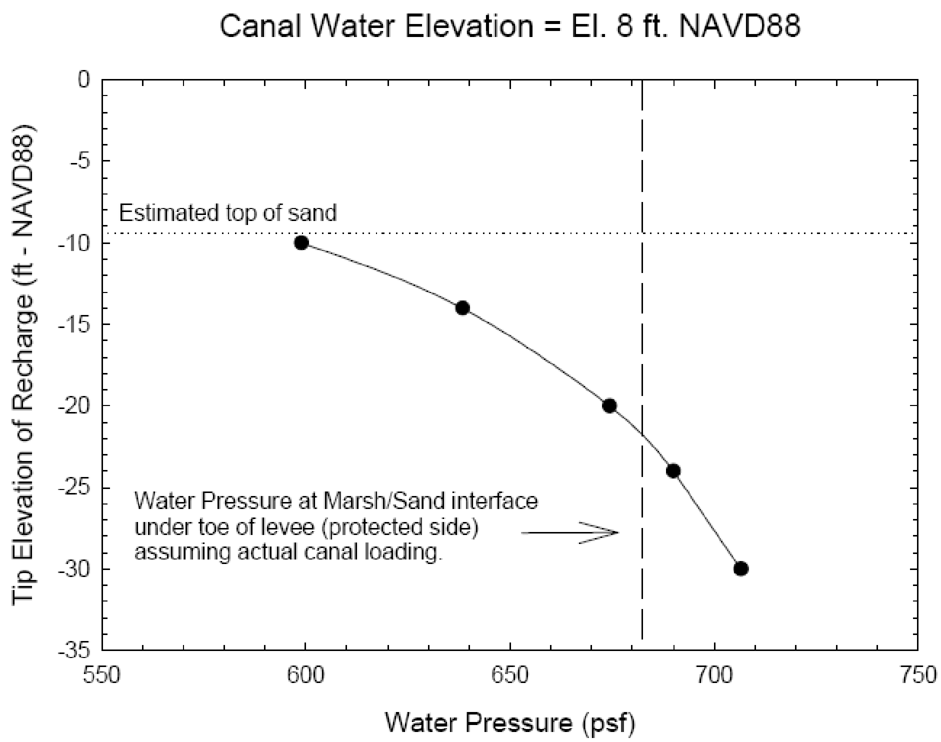


Figure B-20 – Depth of Penetration of Injection Wells for Canal Water at El +6 NAVD for Fully Penetrating Sheetpiling


Canal Water Elevation = El. 8 ft. NAVD88
Tip Elevation of Recharge (ft - NAVD88)
Water Pressure (psf)
Estimated top of sand
Water Pressure at Marsh/Sand interface
under toe of levee (protected side)
assuming actual canal loading.
0
-5
-10
-15
-20
-25
-30
-35
550
600
650
700
750

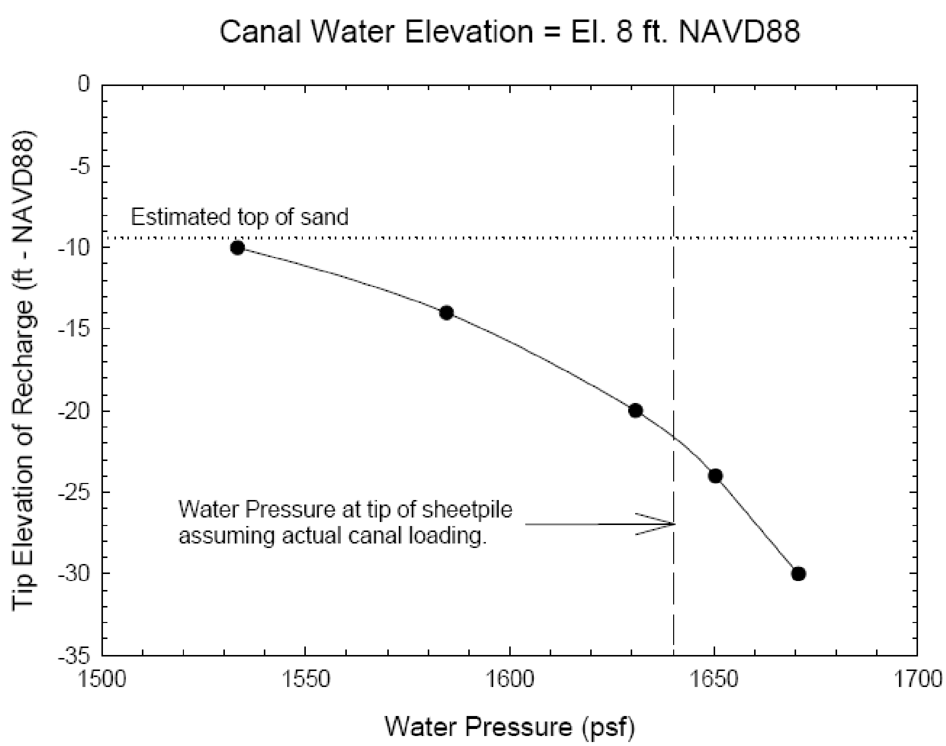

Canal Water Elevation = El. 8 ft. NAVD88
Tip Elevation of Recharge (ft - NAVD88)
Water Pressure (psf)
Estimated top of sand
Water Pressure at tip of sheetpile
assuming actual canal loading.
0
-5
-10
-15
-20
-25
-30
-35
1500
1550
1600
1650
1700

Figure B-21 – Depth of Penetration of Injection Wells for Canal Water at El +8 NAVD for Fully Penetrating Sheetpiling

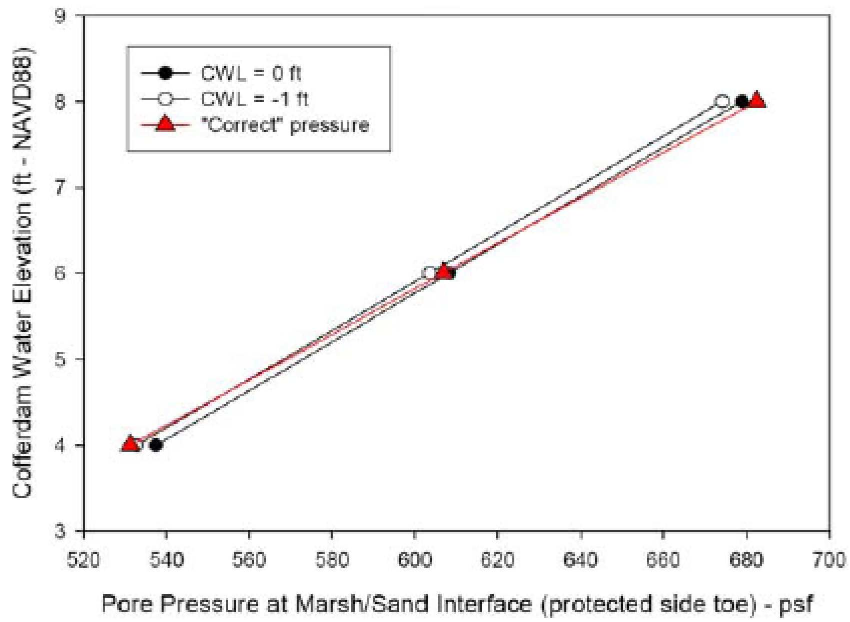


Figure B-22 – Pore Pressures as a Function of Cofferdam Water Elevation for Differing Canal Water Elevations at the Marsh Clay/Sand Interface for Partially Penetrating Sheetpiling

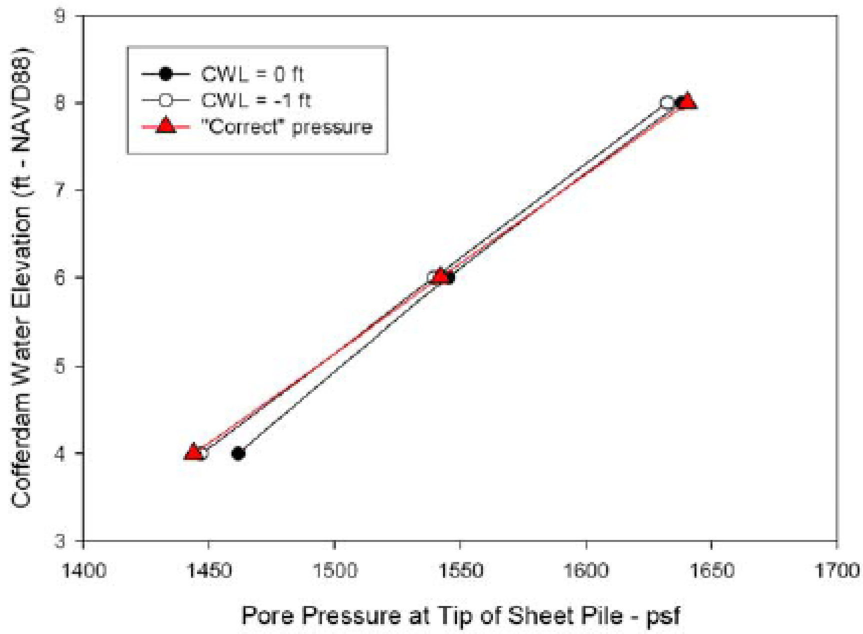


Figure B-23 – Pore Pressures as a Function of Cofferdam Water Elevation for Differing Canal Water Elevations at the Tip of the Sheetpiling for Partially Penetrating Sheetpiling

Table B-1 – Soil Properties

Stratum	**Description (Classification)**	**γ_s (pcf)**	**D_{10} (mm) or w (%)**	**k (cm/sec)**	**Ø' deg**	**c' or s_u (psf)**	**Source of Data**
A	Levee fill (CH)	109		$1.0x10^{-6}$		900	Appendix 8, p. 3-9
		110/**109**		**$1.0x10^{-6}$**		**900**	Appendix 9, p. 5 / **Schwanz**
		110	25-58			837 Ave (440 to 1365)	Test Section
		109		**$1.0x10^{-6}$**		**900**	**Brandon**
B	Swamp/marsh/ organic clay	80 **80**		$1.0x10^{-5}$ **$1.0x10^{-6}$**		300 **300**	Appendix 8, p. 3-9 Appendix 9, p 5 /
	(CH/OH)			**to$1.0x10^{-4}$**		**to 400**	**Schwanz**
	Under levee	172-266					Test Section
	Toe and beyond	54-109					
	In canal	84-203					
		80		**$1.0x10^{-5}$**		**300**	**Brandon**
C	Lacustrine clay			$1.0x10^{-6}$		42	Appendix 8, p. 3-9
	(CH)	**Not considered in analysis**					Appendix 9, p 5 / **Schwanz**
		74				900	Test Section
		Not considered in analysis					**Brandon**
D	Barrier beach sand (SP/SM)	115		$1.5x10^{-2}$	32		Appendix 8, p. 3-9
	-12 to -22 NAVD	118/ **115**		**$1.5x10^{-2}$**	31/ **30**		Appendix 9, p 5 /
	-22 to -35 NAVD	122		$1.5x10^{-2}$	36		**Schwanz**
	-35 to -47 NAVD	118		$1.5x10^{-2}$	32		
	-8 to -22 NAVD		0.088	$1.4x10^{-2}$	29(N)		Test Section
	-22 to -33 NAVD	to 0.130			33(N)		
	-33 to -48 NAVD				29 (N)		

		115		**$1.5x10^{-2}$**	**30**		**Brandon**
E	Bay/sound/	Not considered in analysis					Appendix 8, p. 3-9
	estuarine clay	**102**		**$1.0x10^{-6}$**		**779**	Appendix 9, p 5 / **Schwanz**
	(CH)	100	43-65			900	Test Section
				$1.0x10^{-6}$			**Brandon**

The properties obtained from Appendix 8 (1) were used to perform seepage and stability analyses of the north breach. The properties obtained from Appendix 9 (1) were used in SSI analyses of both the north and south breach. The data identified as Brandon (7,8) and Schwanz (9) were used in their respective analyses of the test section.

Table B-2 – Assessment of Soil Property Data used in Analyses of Test Section

- Levee fill (CH) – The moist unit weight and permeability values for the test section appear reasonable. The undrained shear strength at the test section is slightly lower than assumed by Brandon and Schwanz, but is considered reasonable**.** No modifications are necessary. Additional information is recommended regarding the strength and stiffness of the levee fill on the protected side of the canal. This information will provide site-specific data for assessing the load test results and can also be used in the subsequent assessment of conditions along the entire canal.

- Marsh clay (CH/OH) – The moist unit weight and permeability values for the test section appear reasonable. The undrained shear strength of 300 psf assumed by Brandon appears to be conservative, as it apparently does not allow for an increase in strength below the levee due to consolidation as was done in the SSI analyses presented in Appendix V-9 and by Schwanz. A value of 400 psf was used in the Appendix V-9 analyses and by Schwanz for the soils under the levee. It is recommended that the Brandon analyses consider a sensitivity assessment of the impact of an undrained strength increase under the levee section on the results of his analyses of the test section.

- Lacustrine clay (CH) – This layer was ignored by IPET in their Appendix 8 analyses and also by Brandon and Schwanz. Since the layer is thin and has been shown by IPET to have little impact on the analyses, this assumption appears reasonable.

- Barrier beach sand (SP/SM) – The permeability of the sand stratum seems to have been well characterized using large-scale pump test results. Brandon and Schwanz assumed that the strength of the sand (i.e., friction angle) is similar to that used by IPET in their analyses. The SPT data suggest a relatively constant value of friction angle throughout the stratum to El -22 NAVD. The sand shear strength considered in all of the analyses appears reasonable.

- Bay/sound/estuarine clay (CH) – This layer was ignored by Brandon in his analyses due to its depth and this assumption appears reasonable.